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Sanitary Engineering Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

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2. A summary of approximately 50 words must accompany the paper, a 300-word synopsis must precede it, and a set of conclusions must end it.

3. The manuscript (an original ribbon copy and two duplicate copies) should be double-spaced on one side of 8½-inch by 11-inch paper. Three copies of all illustrations, tables, etc., must be included.

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SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

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Note.—Part 2 of this Journal is the 1961-28 Newsletter of the Sanitary Engineering Division.

The three preceding issues of this Journal are dated January 1961, March 1961, and May 1961.

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DISCUSSION

Storm Water and Combined Sewage Overflows, by Samuel A. Greeley and Paul E. Langdon. (January, 1961. Prior discussion: None. Discussion closed.)
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Journal of the
SANITARY ENGINEERING DIVISION
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SANITARY CONSIDERATIONS OF FIVE-MILE OCEAN OUTFALL

By Charles H. Lawrance, M., ASCE[1]

SYNOPSIS

Engineering considerations are discussed that led to the establishment of the basic and detailed design of the 5-mile submarine effluent outfall at the Los Angeles, Calif. Hyperion treatment plant. Specific and general studies and findings on ocean disposal of sewage effluent that were undertaken as part of this project are outlined. The design of the 5-mile outfall is developed with respect to anticipated dilution, bacterial decline, and shore sampling results.

INTRODUCTION

Existing Plant and Outfall.—The activated sludge plant and 144-in. diameter 1-mile ocean outfall completed at Hyperion, Calif. in 1951 were the culmination of a movement begun in 1943 to clean up the beaches and correct a previously unsatisfactory condition in Santa Monica Bay. The techniques then available to design engineers for the establishment of required outfall lengths were as follows:

1. Float studies to determine current patterns;

2. the equations developed by A. M. Rawn, M. ASCE and H. K. Palmer, F. ASCE[2];

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SA 4, July, 1961.

[1] San. Engr., Hyperion Engrs., currently Proj. Engr., Koebig and Koebig, Inc., Cons. Engrs., Los Angeles, Calif.

[2] "Predetermining the Extent of a Sewage Field in Sea Water," by A. M. Rawn and H. K. Palmer, Transactions, ASCE, Vol. 94, 1930, p. 1036.

3. the equations formulated by W. J. Houghton and H. C. Mason[3];

4. empirical relationships derived from general observations of the performance of existing ocean outfalls in certain cases.

The 7-ft diameter outfall formerly used in conjunction with the Hyperion screening plant had experienced difficulty with leaks and with partial obstruction by sand intruding from the ocean floor, and the screening plant effluent that was discharged had been inadequately treated and resulted in widespread contamination. The existing one-mile 144-in. ocean outfall discharged plant effluent from the high rate activated sludge process, that removed the bulk of the objectionable sewage solids to the point at which the effluent had been physically acceptable in the receiving waters as well as amenable to chlorination as an economical means of bacterial reduction. This outfall was made to terminate in a small diffuser approximately 4000 ft from the surf zone, providing mixing of the chlorinated effluent with the receiving waters at an offshore point. The combined treatment plant and effluent outfall system were designed to meet the state Board of Public Health Salt Water Bathing Standard that prevailed and to restore the beaches and general receiving waters to a satisfactory condition. The facilities were successful in carrying out their mission, all requirements were met and the beaches became increasingly popular.

Recent Developments.—On completion of the Hyperion activated sludge type treatment plant, the state-imposed quarantine of the bathing beaches was completely lifted and the plant operations proceeded in large measure as was anticipated. The continued growth of Los Angeles resulted in a need to expand the sewerage system and the treatment plant, and the mayor appointed a Citizens' Committee to study the sewerage problem. They held hearings into the matter and solicited various engineering opinions including two independent engineering reports by private consultants.[4,5] These independent reports each showed that it would prove most economical to continue to convey the city's sewage to Hyperion for treatment and disposal in Santa Monica Bay and that savings in fixed and operating costs would be enjoyed by expanding only the primary portion of the Hyperion treatment plant and by constructing a long submarine outfall terminating in deep water several miles off Hyperion that would discharge unchlorinated primary effluent through a large diffuser into the cool bottom waters.

The length of the proposed outfall, that was set at approximately 5 miles, was based largely on considerations of bottom topography and of the length needed to reach deep water at which the discharged effluent would become so diluted with the stratified layer of cold and dense bottom water that it would tend to remain submerged. Furthermore, the distance from shore corresponding to the desired depth was approximately equal to the distance between the limits of the contaminated area of the bay and the point of sewage discharge before

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3 "Extension of Green Point Sea Outfall, Capetown," by W. J. Houghton and H. C. Mason, Journal, Inst. of Munic. and County Engrg., London, Vol. 58, September, 1931, p. 471.

4 "Opinions and Comment on Engineering Features of Proposed Solutions of the Sewerage Problem of the City of Los Angeles for Citizens' Committee on Sewerage Problem, George B. Gose, Chairman, September 1954," by Richard R. Kennedy.

5 "Opinions and Comment on Engineering Features of Proposed Solutions of the Sewerage Problem of the City of Los Angeles for Citizens' Committee on Sewerage Problem, George B. Gose, Chairman, September 29, 1954," by Richard Pomeroy.

the days of the Hyperion activated sludge type plant and one mile submarine outfall. Also, the bacteriological aspects of outfall depth, distance, and discharge rate in an equation deduced by Richard D. Pomeroy, F. ASCE[6,7] intended for approximation of operating results, showed general agreement with the arrangements determined by the other means. The proposed system appeared to offer reasonable certainty of success and, indeed some margin for contingency, and the proposals were adopted by the Citizens' Committee and recommended to the City Council for action.

Because of the emergency nature of the overall sewerage problem and the immediate need for facilities expansion, the city contracted for this and other design work, (described in a published paper),[8] with a joint venture of private Los Angeles engineering firms and with general review of the work by a board of consultants composed of R. D. Spencer, Chairman, F. ASCE, D. L. Narver, F. ASCE, I. F. Mendenhall, F. ASCE, F. D. Bowlus, F. ASCE, R. R. Kennedy, F. ASCE, and the City Engineer of the city of Los Angeles represented by L. Aldrich, F. ASCE, and his successor L. A. Pardee, F. ASCE. R. D. Pomeroy, F. ASCE, N. B. Hume, F. ASCE, and M. Butler, F. ASCE acted as advisors.

WATER POLLUTION STANDARDS

The Los Angeles Regional Water Pollution Control Board[9] prescribed standards for the receiving waters to be maintained under future conditions of operation with the proposed 5-mile outfall and 7-mile sludge outfall.[10] Table 1 summarizes the essential requirements of these standards, with the areas indicated in Fig. 1. The specified monitoring program, involving monthly offshore samples and daily beach samples, has been in progress since December, 1957, during which time valuable background data for Santa Monica Bay have been gathered, proving useful in evaluating the 5-mile effluent outfall since its completion in January, 1960 and subsequent operation.

SCHEDULE OF FIELD INVESTIGATIONS

General.—When the engineers undertook the project of design, the city of Los Angeles had barely begun its program of extended offshore studies as described by C. G. Gunnerson, F. ASCE[11] and due to the nature of the sewerage

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6 "Prediction of Probable Bacterial Concentrations Caused by Ocean Sewer Outfalls, Revised December 30, 1953," by R. D. Pomeroy, Report to John A. Carrollo, Relative To Orange County Outfall Under Design.

7 "The Empirical Approach for Determining the Required Length of an Ocean Outfall," by Richard Pomeroy, Proceedings, 1st Internatl. Conf. on Waste Disposal in the Marine Environment, Pergamon Press, New York, July, 1959.

8 "Long Land Outfall Built through Baldwin Hills," November, 1957, p. 782; "Hyperion Plant Expanded to Treat 420 MGD," December, 1957, p. 856; and "Two Long Ocean Outfalls Constructed," January, 1958, p. 6, by D. L. Narver, Jr. and E. H. Graham, Jr., Civil Engineering.

9 Resolution No. 56-44, Los Angeles Regional Water Pollution Control Bd., Los Angeles, Calif., November 29, 1956.

10 Currently in use for submarine disposal of plant effluent and digested sludge, respectively.

11 "Sewage Disposal in Santa Monica Bay, California," by C. G. Gunnerson, Transactions, ASCE, Vol. 124, 1959, p. 823.

TABLE 1.—CITY OF LOS ANGELES MONITORING PROGRAM STANDARDS REQUIRED[a]

Item	Sampling Stations			
	A	B	C	Shore
Santa Monica Bay Area	1	2	2	3
Range of distances from origin feet	< 5000 to < 7500	13000 to 22000	23000 to 42000	1500 ft. from H.W.L.
Coliform organisms MPN per milliliter	no limit	geometric mean of B & C < 100		10
Dissolved oxygen	$\leq$ 6 PPM	—	—	—
Grease, average	$\leq$ 2 PPM	—	—	—
Floating or suspended sewage solids	None	None	None	None Below H.W.L.
Transparency, feet by secchi disk	no limit	$\geq$ 20	$\geq$ 20	—
Organic content of bottom samples	no limit	$<$ 50%	$<$ 50%	—
Annual increase of	no limit	$<$ 3%	$<$ 3%	—

[a] By May 2, 1956 order of the state Water Pollution Control Board

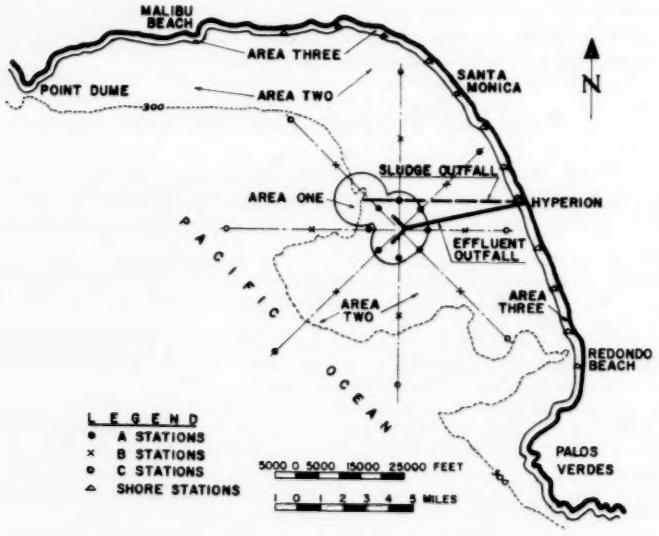

FIG. 1.—RECEIVING WATERS POLLUTION CONTROL STANDARDS FOR HYPERION SLUDGE AND EFFLUENT OCEAN OUTFALL DISCHARGES

emergency, these studies and others described elsewhere by the writer[12], and by the engineers[13], proceeded simultaneously with preliminary investigations and design. The basic field investigations were concerned with ocean currents and decline of sewage bacteria in sea water. The initial bacteriological and oceanographic findings indicated that there was less margin for safety than originally believed; therefore, the bacteriological aspect of the program was greatly expanded and carried on even into final design. The final results gave reasonable reassurance of success, including a margin of safety.

Scope.—In addition to the vital bacteriological aspect and the study of currents, the establishment of time of travel of pollutants and time for bacterial decline, required the investigation of other aspects of oceanography and effects of discharge on marine resources.

TABLE 2.—OCEAN OUTFALLS STUDIED DURING DESIGN PERIOD OF NEW FIVE-MILE HYPERION OCEAN OUTFALL FOR EFFLUENT DISPOSAL

Locality (1)	Owner (2)	Size, in inches (3)	Length from Shore, in feet (4)	Terminal Depth Average Ports (5)	Initial Dilution (6)	Discharge MGD (7)
Santa Monica Bay	City of Los Angeles	144	4400	45	Up to 20	230
San Pedro Channel	Los Angeles County Sanitation Dists.	60	5000	105	170	50
San Pedro Channel	Los Angeles County Sanitation Dists.	72	6500	155	290	110
Orange County Coast	Orange County Sanitation Dists.	78	7000	55	26	24

The areas of field investigation included three general ocean disposal sites off the coast of Southern California as described briefly in Table 2. The two areas outside of Santa Monica Bay were selected because they were fairly close by and involved substantial primary effluent flows through major outfalls. The San Pedro Channel site provided very deep water and the Orange County Coast site provided oceanographic conditions similar in many ways to those of Santa Monica Bay.

Oceanographic Consultants.—A complete and exhaustive oceanographic investigation, embracing an entire year, in order to cover cyclic aspects, was

12 Discussion by Charles H. Lawrance and David R. Miller of "Sewage Disposal in Santa Monica Bay, California," by C. G. Gunnerson, Transactions, ASCE, Vol. 124, 1959, p. 823.

13 "Ocean Outfall Design," Hyperion Engineers, A Joint Venture, Holmes & Narver, Inc., Daniel, Mann, Johnson & Mendenhall; and Koebig & Koebig, October 15, 1957.

undertaken by the Allan Hancock Foundation of the University of Southern California, Los Angeles, Calif. This foundation specializes in oceanographic research. The work of the Hancock Foundation was begun in June, 1955, and completed in August, 1956. Their final reports on oceanography were submitted in September, 1956.[14,15,16,17]

Bureau of Sanitation.—The Bureau of Sanitation of the city of Los Angeles conducted oceanographic investigations in Santa Monica Bay from August, 1954 to October, 1956 to gather basic data for use in connection with the design of the expanded facilities at the Hyperion treatment plant, and in the operation of both the existing and proposed works. The findings have been reported by Gunnerson[12] and elsewhere.

OCEANOGRAPHIC FINDINGS

Description of Santa Monica Bay.—Santa Monica Bay is depicted in Fig. 1. Except for 3 miles of precipitous cliffs above a narrow, rocky beach cut into the Palos Verdes Hills, the entire bay coastline is typically sandy beach, excellent in character, well developed, and heavily used for recreation.

The bottom of the bay slopes gently from the shore to a depth of about 300 ft, forming a shelf of from 3 miles to 6 miles wide. There are two semi-diurnal tides of about 3.5 ft average range. Tidal and other currents are relatively weak and random. Occasional heavy swells from far off in the Pacific Ocean are received, but the more common waves result from coastal sea breezes that are generally light and variable in direction although their greatest strength and direction is normally onshore. The Hyperion outfalls are the only known source of sewage effluent discharging into the bay. Two existing subaqueous recirculation water systems for steam power plants at El Segundo and Redondo Beach, respectively, act in combination with the existing Hyperion discharge presently tend to form a warm water wedge that reduces some of the onshore water circulation from parts of the outer bay.

The area one might expect to be involved generally with the Hyperion discharge lies roughly between the cities of Redondo Beach and Santa Monica, Calif. Although the volume of water over this area of the continental shelf is about 12,000 times the present daily volume of sewage, the actual available volume of water for diluting the outfall's discharge is far less than this value. This is true because of the relatively weak currents prevailing and relatively low rate of interchange of water fresh from the Pacific Ocean with water of the bay.

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14 "The Oceanography of Santa Monica Bay, California," by Robert E. Stevenson, Richard B. Tibby, and Donn S. Gorsline, a Final Report submitted to Hyperion Engineers by the Geology Dept., Univ. of Southern California, September 18, 1956; "Studies on Coliform Bacteria Discharged from the Hyperion Outfall, Final Bacteriological Report," by Sydney C. Rittenberg, a Final Report submitted to Hyperion Engineers by the Univ. of Southern California, August 29, 1956.

15 "Contributions to a Biological Survey of Santa Monica Bay, California," by Olga Hartman, a Final Report to Hyperion Engineers by the Geology Dept. of the Univ. of Southern California, September 17, 1956.

16 "Plankton and Associated Nutrients in the Water Surrounding Three Sewer Outfalls in Southern California," by Robert E. Stevenson and John R. Grady; a Final Report submitted to Hyperion Engineers by the Geology Dept. of the Univ. of Southern California, September 4, 1956.

17 "Submarine Geology of Santa Monica Bay, California," by Richard D. Terry, Stuart A. Keesling, and Elazar Uchupi, a Final Report submitted to Hyperion Engineers by the Geology Dept. of the Univ. of Southern California, September 11, 1956.

Design Objectives of Oceanographic Investigations.—The general objective of the oceanographic investigations was to determine that location of the outlets from the outfall and diffuser system that would provide the optimum balance of economy and reliable performance in utilizing the receiving waters for dilution and dissipation of the discharge within the prescribed water pollution control standards. Dilution of the discharge in the initial stage is determined by diffuser characteristics and ocean currents. Subsequently, dilution is established by many complex physical phenomena including water mass movement, turbulence, and wave action. These phenomena and the factors affecting bacterial decline must be studied for the individual receiving waters from which it is possible to prepare a logical design.

Nature of Oceanographic Measurements.—Current meters were used to determine instantaneous velocities and directions, and float studies, drift cards, thermal surveys, and dye and tracer studies were used in evaluation of the net water motion in Santa Monica Bay. The work has been described elsewhere.[13,14,15] A sample bathythermogram is shown in Fig. 2, which also includes a sample temperature profile of the shelf water mass in Santa Monica Bay on a line extending normally from Hyperion. A generalized circulation pattern for spring and summer developed by the Hancock Foundation is shown in Fig. 3. A frequency distribution of surface currents passing the general proposed diffuser area 5 miles off Hyperion and traveling in a direction so as to intersect the shoreline between the northerly limits of Santa Monica and the southerly limits of Redondo Beach is depicted in Fig. 4. (Average velocity was 31.0 fpm. Origin is 5 miles offshore. Critical area taken to be sector from northern limits of Santa Monica to southern limits of Redondo Beach, corresponding to azimuths of 10° to 120° from origin.) The analysis and diagram were the work of the Bureau of Sanitation.

SANITARY RESEARCH AND INVESTIGATIONS

General.—This work combined the considerations of physical dilution of sewage effluent and bacteria within the diluted mass as applied to specific ocean outfalls and receiving waters. Extensive field measurements were made for the three major ocean outfalls described in Table 2. In addition, performance data were obtained for all major ocean outfalls for which offshore measurements were available. The data were analyzed for the purpose of confirming methods applied for the Hyperion outfall design. The various formulas for initial and subsequent dilution and of bacterial decline of sewage effluent discharged from submarine outfalls were applied insofar as possible both to the field data locally obtained and the data from outfalls located elsewhere. A reasonable appraisal of the merit of these formulas was possible.

Field Measurements.—The bulk of the investigations involved measurements of the decline of the sewage bacteria in sea water following discharge from existing ocean outfalls described in Table 2. Laboratory data were available for the various effluents being discharged, particularly for the Hyperion plant effluent. Coliform concentrations of initially diluted effluent were measured at or near the surface at the point of emergence of the effluent field and samples were taken at timed intervals along the path of the effluent field as it moved away from the area above the outlets. Frequently salinity measurements, temperature measurements, or ammonia measurements, or all three were made, thereby providing a means of computing physical dilution. Identification of the

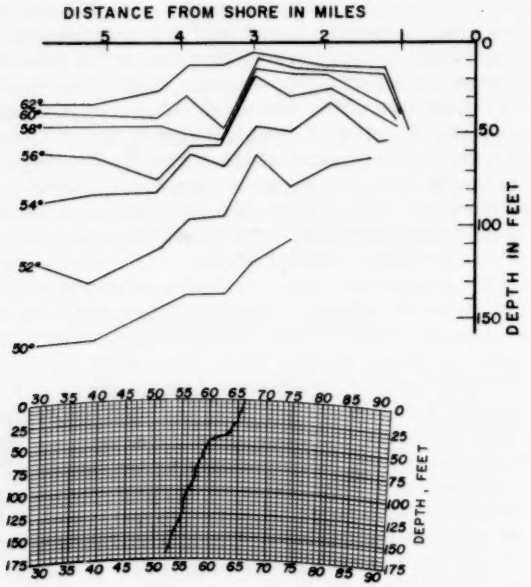

FIG. 2.—SAMPLE BATHYTHERMOGRAM AND TEMPERATURE PROFILE IN SANTA MONICA BAY NEAR NEWOUTFALL LOCATION

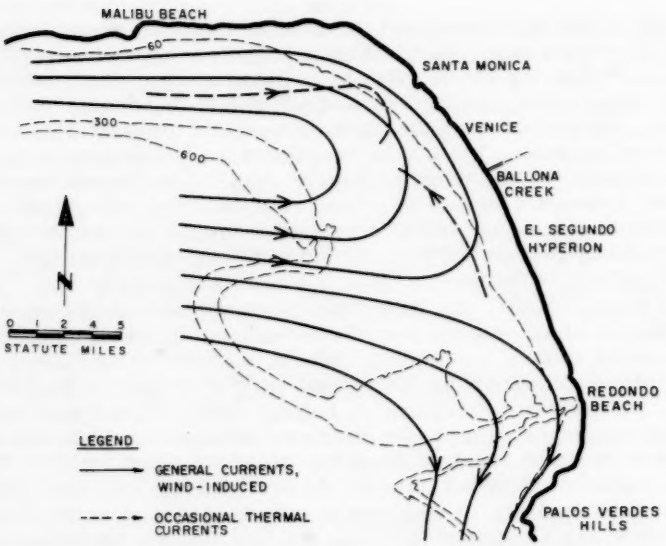

FIG. 3.—GENERALIZED FLOW PATTERN FOR SANTA MONICA BAY DURING SPRING AND SUMMER WEATHER

effluent field was accomplished by means of floats as well as sea-marker dye. Samples were collected at depth on several occasions providing data from which profiles of bacterial concentrations could be constructed. On a few occasions an areal grid pattern of sampling covering several thousand square feet was accomplished for surface concentrations of coliform bacteria within the effluent field.

The result of many observations for a given outfall and effluent were compiled, studied and compared with those for the other receiving waters and outfalls under study. A special 24-hr offshore study utilizing radioactive scandium-46 as a tracer in the Hyperion plant effluent served to verify the "dye patch" technique as a valid method of "tagging" the effluent field, to measure relatively high physical dilution levels, and to demonstrate the effects of non-dilution phenomena, particularly sedimentation, as a cause of bacterial decline in an effluent field. The results of these various studies have been published by Gunnerson[12], the Hancock Foundation[15,16,17,18] and the engineers[14]. The engineers supplemented these studies by analysis of performance of various other major ocean outfalls as subsequently described. The object of all these investigations was to develop design parameters insuring adequate physical dilution of the discharge and most particularly adequate bacterial reduction.

Additional Performance Data.—Offshore bacterial data or chemical data or both were obtained from a number of ocean outfalls representing a variety of effluents, outfall conditions, and receiving waters. These included the following: the Deer Island, Moon Island and Nut Island Outfalls in Boston Harbor[19]; the New Bedford Outfall in Buzzards Bay[20]; The Mamaroneck and New Rochelle Outfalls, in Long Island Sound, N. Y.[21,22]; the Fort Lewis Outfall in Puget Sound, Washington[23]; the International Sewer into the Pacific Ocean near Tiajuana, Mex.[24]; and the Sand Island, Ala Moana, and Ward Avenue Outfalls of Honolulu, Hawaii.[25] Table 2 indicates the time of measurements, condition of discharge, nature of data, and nature of data analysis. Some of the receiving waters estuarine, and some were open coastal waters with strong tidal currents or prevailing currents. These contrast with the receiving waters for which field data were obtained and analyzed (open coastal type with random currents typically). The latter are also included in Table 3.

Application of Formulas.—The initial dilution experienced by a subaqueously injected stream of sewage effluent during its rise through the surrounding ocean

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18 "Summary Report, Oceanographic Investigation of Santa Monica Bay," Bureau of Sanitation, Los Angeles, Calif., 1956.

19 Report of the Special Commission on the Investigation of the Discharge of Sewage into Boston Harbor, Mass. Dept. of Pub. Health, House No. 1600, December, 1936.

20 "Preliminary Studies on the Viability and Disposal of Bacteria in the Sea," by B. H. Ketchum, C. L. Carey, and M. Briggs, Limnology, Water Supply, and Waste Disposal, Amer. Assn. for the Advancement of Science, 1949.

21 "Radium as a Tracer in the Sewage Flow Measurements," by T. G. Bullen and W. F. O'Connor, Sewage and Industrial Wastes 26: 497-507, April, 1954.

22 "Radon in Sewage Outfall Studies," by T. G. Bullen and W. F. O'Connor, Sewage and Industrial Wastes 26: 627-34, May 1954.

23 "An Investigation of Pollution in the Vicinity of the Fort Lewis Sewer Outfall," Technical Bulletin No. 10, State of Washington Pollution Control Commission, June, 1951.

24 "Travel of Pollution in Shoreline Coastal Waters," by I. Nusbaum, H. E. Miller, and A. W. Reinhardt, Proceedings, ASCE, Vol. 81, August, 1955, p. 781.

25 Report on Ocean Current Studies for Sand Island Outfall Sewer, Honolulu, Oahu, T. H., City and County of Honolulu, Hawaii, July 2, 1951.

TABLE 3.—OUTLINE OF STUDIES OF OCEAN OUTFALL PERFORMANCE

No.	Name of Outfall	Type of Outlet	Type of Effluent	Nature of Performance Data				Period of Data	Computed, Theoretical and/or by Data					Formulas Applied to Data				
				Chemical	Bacterial	Beach	Offshore		Init. Dil'n.	Subseq. Dil'n.	Geom. Mean	Log Prob'y	Gross T-90	Rawn & Palmer	E. J. Conway	N. B. Brooks	R. D. Pomeroy	Rawn, Brooks & Bowerman
(1)	Former Hyperion	1-45° Upward	Fine Screen, Part. Chlor.		x	x		Fall, 1947 & 1948	x					x	x	x		x
(2)	One-Mile Hyperion	8 Diffuser Ports	High-Rate Act. Sludge	x	x	x	x	Within 3/51-1/56 †	x	x	x	x		x	x	x		x
(3)	Orig. Whites Point (2)	Open-End	High-Rate Primary		x	x		1/52-1/53 †	x		x	x		x	x	x		x
(4)	Ext. Whites Point (2)	Diffusers	High-Rate Primary	x	x	x	x	4-54-12/54	x	x	x	x			x	x		x
(5)	90" & 72" Whites Point	Diffusers	High-Rate Primary	x			x	11/2/56	x									x
(6)	Former Orange County	Open-End	Chlorin'd Primary		x	x		11/48-11/49	x		x	x		x				
(7)	Present Orange County	Small Diffuser	Unchlor. Primary	x	x	x	x	Within 7/54-1/56	x	x	x	x		x	x	x	x	
(8)	Deer Island	Small Diffuser	Coarse Screened	x	x		x	1935-36 Flood Tide	x	x			x	x	x	x		x
(9)	Moon Island	Open-End	Coarse Screened	x	x		x	1935 Ebb Tide	x	x			x	x	x	x		x
(10)	Nut Island (1 or 2)	Vertical, Open-End	Coarse Screened	x	x		x	1915 & 1935-36	x	x			x	x	x	x		x
(11)	New Bedford	Vertical, Open-End	Sewage	x	x		x	1915 & 1946 †	x	x				x	x	x		x
(12)	Mamaroneck and New Rochelle	Sep. Open-End Outfalls	Plant Effluent	x			x	1951	x	x				x				
(13)	Fort Lewis	Open-End	Raw Sewage		x		x	1951					x					
(14)	International	Open-End	Virtually raw		x	x		1954					x					
(15)	Ala Moana Ward Avenue	Open-End, (Both)	Raw Sewage (Both)		x	x	x	1952	x	x					x	x		x

Notes:
1. These studies are elaborated in Ocean Outfall Design, by Hyperion Engineers, a Joint Venture, October 15, 1957.
2. Pomeroy Formula was applicable only in one case.

waters has been the subject of considerable study. The subsequent physical dilution and bacterial decline of the initially-diluted mass or "sewage field" is dependent to a considerable degree on the initial dilution and is therefore discussed concurrently with initial dilution. Less study has been devoted to subsequent dilution and bacterial decline than to initial dilution.

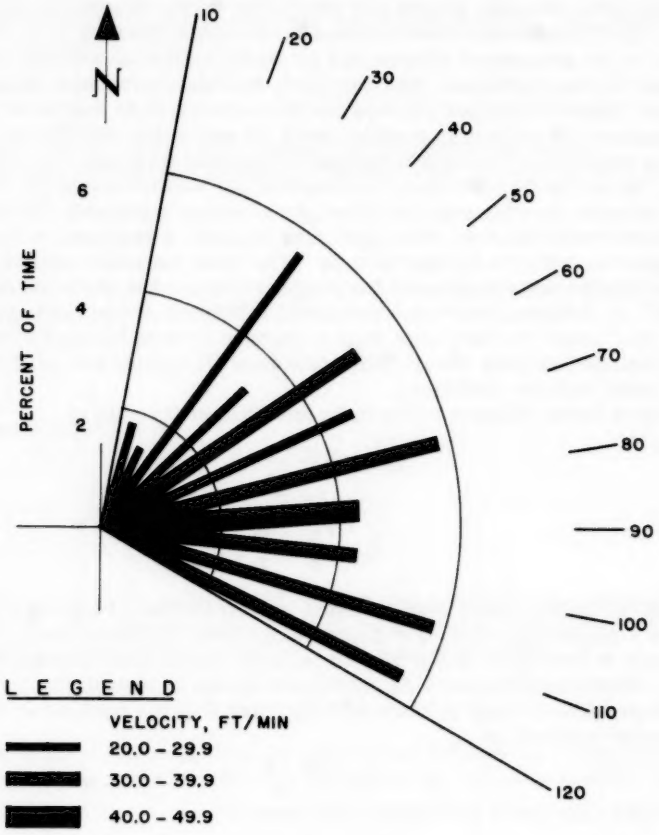

FIG. 4.—DISTRIBUTION OF CRITICAL ONSHORE CURRENTS FROM DISPOSAL SITE

The formulas for initial dilution of sewage effluent on submarine injection and for subsequent physical dilution and bacterial decline have been compiled by the engineers[14], and by E. A. Pearson[26]. The authors of these formulas

26 An Investigation of the Efficacy of Submarine Outfall Disposal of Sewage and Sludge, for the State Water Pollution Control Board, Berkeley, Calif., by E. A. Pearson, December, 1955.

include Rawn and Palmer[2]; Houghton and Mason[3]; E. J. Conway[27]; R. D. Pomeroy[6]; Rawn, F. R. Bowerman, and N. H. Brooks[28]; and Brooks[29]. The well-known Rawn and Palmer formulas cover both initial dilution and subsequent dilution and bacterial decline although they were not designed to apply to very large diffusers. The original Rawn and Palmer data for initial dilution as reanalyzed by Rawn, Bowerman, and Brooks utilizing hydraulic similitude have furnished valuable graphs for predicting initial dilution for all kinds of outlets. The Houghton and Mason formula covers initial dilution only. Conway's formula is for subsequent dilution and bacterial decline as are also the Pomeroy and Brooks formulas. The Houghton and Mason formula is similar to Rawn and Palmer's and permits application to brackish as well as to salt water. Because the outfalls in question were all salt water outfalls and because the Rawn and Palmer formulas had graphs to expedite their use, the latter were used by the writer in preference to Houghton and Mason's formula.

The current flux formula was also applied where applicable for the determination of initial dilution. This particular formula is presented subsequently in conjunction with the Brooks formula. The basic formulas with their effect of initial dilution and components for physical dilution and bacterial decline or "die-off" as follows: Formulas for initial dilution of stream sewage effluent during rise from the discharge point to surface or vertical equilibrium point will be included because sewage field spread and dissipation are partly dependent on initial dilution conditions.

Effect of Initial Dilution.—The Rawn and Palmer formula is

$$S_o \sim Q^{-0.61} L^{2.35} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and

$$X_o = \frac{L}{8} + \frac{D}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which S_o is the initial dilution which is total volume of sea water-effluent mix/one volume of effluent; Q is the discharge rate of effluent and L refers to the length of travel of rising cone or jet from submerged outlet to the ocean surface of intermediate point; X_o denotes the radius of virtual ring from which the sewage effluent "field" spreads laterally; and D is the port outlet diameter. The Conway equation is

$$S_o \sim Q^{-1} L^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The Brooks equation is the current flux formula;

$$S_o = \frac{U b h}{Q} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

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27 "Biochemical Surveys," by E. J. Conway, Report on the Condition of the Sea Water Resulting from the Present West Pier and Brandy Hale Sewage Discharges Dun Laoghaire; private Report of investigation by E. J. Conway, Prof. of Bacteriology, University College, Dublin, Ireland.

28 "Diffusers for Disposal of Sewage in Sea Water," by A. M. Rawn, F. R. Bowerman, and N. H. Brooks, Proceedings, ASCE, March, 1960, Vol. 86, No. SA 2.

29 "Diffusion of Sewage Effluent in an Ocean Current," by N. H. Brooks, Proceedings, 1st Internatl. Conf. on Waste Disposal in the Marine Environment, Univ. of California, Berkeley, Calif., July, 1959.

in which U is the current velocity (assumed uniform), b denotes the width of a plume-like sewage effluent "field" at a "nominal" distance downstream from the "boil" or the diffusers, and h (or P) refer to the depth of the sewage effluent field. In the Pomeroy equation

$$S_o \sim Y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which Y denotes the submergence of the discharge outlet. The Rawn, Bowerman, and Brooks equation is

$$S_o = f\left(F\frac{Y}{D}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which F denotes the Froude number for the discharge outlet.

Component of Physical Dilution.—The Rawn and Palmer formula is

$$\frac{S_x}{S_o} = \frac{X}{X_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in S_x or S_R or S_t refers to subsequent dilution at x distance or R radius or t time away from the surface "boil." The Conway equation is

$$\frac{S_R}{S_o} = 10^{K_2 R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

The Brooks equation is

$$\frac{S_t}{S_o} = \frac{1}{\operatorname{erf}\frac{b}{\sqrt{\epsilon\, t}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which erf denotes the standard error function and ϵ is the diffusion coefficient. In Eq. 9 S_t is taken in the center line of field and prepresents the maximum concentration of the pollutants. The Pomeroy equation is

$$\frac{S_x}{S_o} \sim X^{1.5^{\pm}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Component of Bacterial Die-Off.—Rawn and Palmer have supplied no actual equation but have noted 10 coliforms per ml as equivalent to $S_x \approx 225$. The Conway equation is

$$\frac{C_o}{C_R} = 10^{K_2 R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

in which C_o denotes the concentration of coliforms in "boil" or at the origin and C_R is the concentration at R radius from the "boil" or origin. The Brooks equation is

$$\frac{C_o}{C_t} = 10^{\frac{t}{T}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

in which T is the time for 90% bacterial decline of any selected portion of the sewage effluent field, and C_t is the coliform concentration after time t travels

from the "boil" or origin. In Eq. 12 C_t represents the maximum bacterial concentration. The Pomeroy equation is

$$\frac{C_o}{C_t} \sim X^{\pm 0.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

Components Combined; Overall Formulas.—The Rawn and Palmer equations are

$$t = \frac{2}{3\,F} X^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

$$P = \frac{L}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (15)$$

and

$$F = \frac{Q\,S_o}{2\,P\,X_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (16)$$

in which F is the factor for sewage field spread. The Conway equations are

$$\left(\frac{S_F}{S_o}\right) = \left(\frac{C_o}{C_R}\right) = 10^{KR} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (17)$$

or

$$\log \frac{C_o}{C_R} = KR \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (18)$$

The Brooks equation is

$$\left(\frac{S_t}{S_o}\right)\left(\frac{C_o}{C_t}\right) = \frac{10^{t/T}}{\operatorname{erf} \frac{b}{\sqrt{\epsilon\, t}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (19)$$

The Pomeroy equation is

$$N_{80}\,\frac{K\,Q^2}{Y\,X^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (20)$$

in which N_{80} denotes the eighty-percentile or coliform concentration value that will not be exceeded more than 20% of the time. It is of significance in the California Salt Water Bathing Standard.

The engineer's report[14] contains considerable illustrative data on the application of Eqs. 1 through 20 to actual outfall performance data.

The formulas for physical dilution, both initial and subsequent, and for bacterial decline are empirical at least to a degree. The greatest differences among the formulas appear in the formulas for spread and dissipation of sewage fields. Some of these have rigorous derivation of at least a portion, and all require use of empirical coefficients. The Pomeroy formula, Eq. 20, has a single coefficient and is intended to yield only approximate results. The Rawn and Palmer formulas are "slanted" towards locating the position at which a physical dilution of approximately 225 is reached; at this point it is alleged the

bacterial decline will have been sufficient to bring compliance with the California Salt Water Bathing Standard of not more than 10 coliforms per ml more than 20% of the time. The Conway formula, Eq. 17, is a monomolecular equation describing a curve of least fall of sewage effluent concentration under the influence of tidal currents but is also purported to be applicable to shore concentrations in the absence of a shelving beach. Brooks' formula, Eq. 19, is designed primarily for large submarine diffusers yielding deep sewage fields from which subsequent diffusion is exclusively horizontally as the field moves

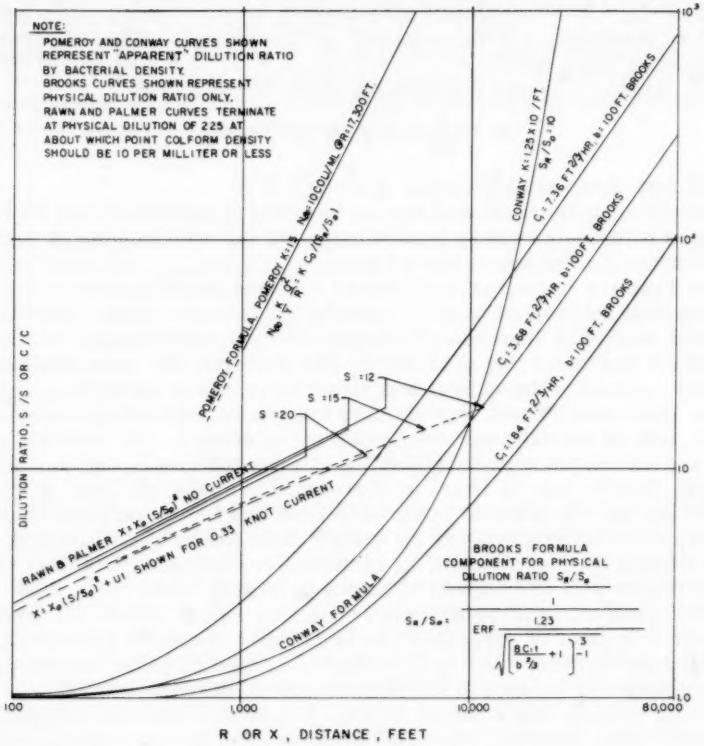
NOTE:
POMEROY AND CONWAY CURVES SHOWN REPRESENT "APPARENT" DILUTION RATIO BY BACTERIAL DENSITY.
BROOKS CURVES SHOWN REPRESENT PHYSICAL DILUTION RATIO ONLY.
RAWN AND PALMER CURVES TERMINATE AT PHYSICAL DILUTION OF 225 AT ABOUT WHICH POINT COLFORM DENSITY SHOULD BE 10 PER MILLITER OR LESS
POMEROY FORMULA, POMEROY K=1.5 Ns=1000 COLI/ML @ R=1.7 300 FT
CONWAY K=1.23 x 10⁻⁵ /FT
Sa/So=10
C=7.36 FT²/³/HR. b=100 FT. BROOKS
C=3.68 FT²/³/HR. b=100 FT. BROOKS
C=1.84 FT²/³/HR. b=100 FT. BROOKS
S=12
S=15
S=20
RAWN & PALMER X=Xo(S/So)^4 NO CURRENT
X=Xo(S/So)^4 + UI SHOWN FOR 0.33 KNOT CURRENT
CONWAY FORMULA
BROOKS FORMULA COMPONENT FOR PHYSICAL DILUTION RATIO Sa/So
DILUTION RATIO, S/S OR C/C
10³
10²
10
1.0
100
1,000
10,000
80,000
R OR X, DISTANCE, FEET

FIG. 5.—ILLUSTRATION OF MAJOR FORMULAS FOR SPREAD AND DISSIPATION OF SEWAGE FIELD IN SEA WATER

under the influence of a unidirectional current. The Brooks formula contains an exponential component similar to the Conway formula.

For the purposes of this paper an illustration is presented for a single hypothetical outfall condition to which the four major formulas are applied. Fig. 5 is a chart depicting the four general equations describing the spread of a sewage field, namely those of Rawn and Palmer, Pomeroy, Brooks and Conway. The Pomeroy and Conway curve shown in Fig. 5 represent "apparent" dilution

ratio by bacterial density. The Brooks curves shown represent physical dilution ratio only. The Rawn and Palmer curves terminate at physical dilution of 225 at about which point coliform density should be ≤ 10 per ml. In order to reduce them to a common denominator, the following assumptions have been made reflecting reasonable conditions often actually experienced: Q = 100 mgd, discharge rate; Y = 50 ft, outlet depth; U = 0.33 knot = 0.55 fps, average current; S_O = 20 initial dilution; b = 100 ft, initial field width in Brooks formula; X_O = 15 ft, radius of "virtual ring." (Other values used correspond to the other values assumed for initial dilution in the Rawn and Palmer curves.); F = 20, factor in computing time to reach dilution 225 in Rawn and Palmer formulas; C_O = 20,000 coliforms per milliliter, average concentration in "boil"; N_{80} = 80 percentile = 5 N_{50}, assumed for this example; N_{50} = 50 percentile or geometric mean density of surf sample; T = T-90 = 4 hr per 90% decline, overall, due to all causes including turbulent diffusion; and

$$K = \frac{1}{UT} = 1.25 \times 10^{-4} \text{ per ft},$$

overall rate of bacterial decline = Conway's K

It should again be mentioned that the formulas in question do not all involve the same criteria, as will be pointed out in the following comments regarding the individual curves appearing in Fig. 5.

The Pomeroy curve, as such, shows only that bacterial decline occurs as the square of the distance from the outlet in the case of random currents. The end point, where the 80-percentile is reduced to 10 coliforms per ml, is actually off the plot, occurring at 17,300 ft. The Pomeroy "K" is an empirical coefficient applying to the effluent and receiving waters in question.

The Rawn and Palmer curves show that physical dilution proceeds as the square root of the distance under quiescent conditions. The coliform organisms are not related to physical dilution directly but only to the "end point" of physical dilution that is taken to correspond to a coliform level of not more than 10 per ml. The distance required to meet this "end point" under quiescent conditions varies inversely as the square of the initial dilution. When a current condition is superimposed, the additional downcurrent travel is increased in accordance with interrelated equations previously noted (Eqs. 1-20).

The Conway curve is distinguished by its exponential nature. It represents a condition typical of "apparent" or bacterially measured dilution ratio. In order for the coliform count to be reduced to 10 per ml under the assumptions of the example, a distance of 26,400 ft is theoretically required.

Brooks formula curve shown in Fig. 5 is restricted to the component for physical dilution ratio due to turbulent diffusion. The bacterial decline element, which is not shown, is similar in nature to that of Conway. If the true bacterial decline rate were 4 hr for 90% decline by all causes except dilution by turbulent diffusion, then the physical dilution curve of the Brooks formula would be directly additive to the Conway curve shown because of the particular values used in the example.

The Conway formula has been of particular interest to the writer because it has provided a ready basis of comparison among many differing outfall conditions with either actual, chemically-measured physical dilution or "apparent" bacterially-measured overall dilution. Theoretically the logarithm of the dilution ratio is proportional to the distance from the outlets, and the value of the constant "K" should not change with distance. Actually, however, Conway's

"K" has been found to decrease with distance in a most consistent fashion as illustrated in Fig. 6, showing the best fit curves for ten different outfalls representing a variety of discharge conditions and receiving waters. These curves include both chemical and bacterial measurements and involve both direct downstream distances in the path of well-defined tidal currents and oblique distances from an offshore coastal outlet to a beach station with random currents intervening. The values of the exponent "K" vary widely among the various curves of each category of measurement, thereby discouraging the direct transposition of the results from one area to another area. Where there is an appreciable difference between values of initial dilution for outfalls discharging into the same receiving waters, the higher initial dilution condition always shows a lower value of "K" at a given distance.

Because the curves all generally resemble equilateral hyperbolas and suggest asymptotic values of both ordinate and abscissa, the writer attempted to fit these curves with a general equation by substituting Conway's formula into the general equation of an equilateral hyperbola;

$$(K - K_o)\ (R - R_o) = \frac{a^2}{2} \quad \ldots\ldots\ldots\ldots\ (21)$$

in which K_o, R_o, and "a" are constants describing the curve.
Then by substituting Conway's formula

$$\log\left(\frac{C_o}{C_R}\right) = K\,R \quad \ldots\ldots\ldots\ldots\ (18)$$

Eq. 21 becomes

$$\log\left(\frac{C_o}{C_R}\right) = \frac{a^2}{2}\,\frac{(R)}{(R-R_o)} + K_o\,R. \quad \ldots\ldots\ldots\ldots\ (22)$$

In order that Eq. 22 have meaning, it is necessary that R_o be negative in sign, that is, that the curve must regress to an origin of minus time or minus distance, because distance and time are equivalent for the presumed current conditions. This is not in conflict with the concept of an outfall versus a true point source in developing diffusion, and it has been found that the curves of Fig. 6 did show origins of minus value. Four such curves are depicted in Fig. 7 and satisfactory curve fits were possible for these and some of the other curves. However, the curve fitting in general was not sufficiently promising to warrant presentation of Eq. 22 for other than illustrative purposes.

Eq. 22 is of a general, empirical nature describing dilution ratio, either "apparent" by bacterial measurement or physical by chemical measurement, for a sewage effluent field as it travels away from the outlet(s) under the influence of either tidal currents or random currents, as measured for a variety of receiving waters both at shore points and offshore points.

It will be noted that Eq. 22 embodies two components. Provided R_o is always negative, the first element provides for an initial rapid increase of dilution ratio that proceeds at an ever decreasing rate, eventually reaching a virtual ceiling.

The second is an exponential component identical in form to Conway's "K" but only vestigial in degree, suggesting long term tendency for continued decline of the discharge, both chemical and bacterial, and even continued increase

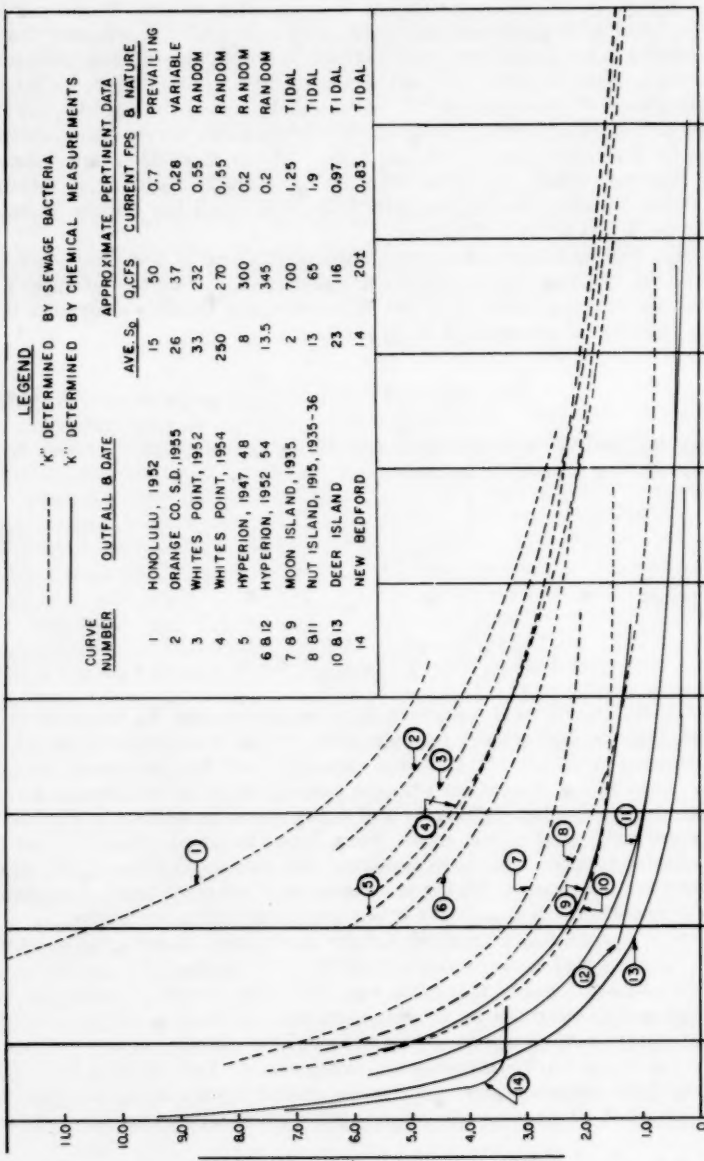

FIG. 6.—BEST FIT CURVES OF CONWAY'S "K" VERSUS DISTANCE FROM OUTLETS FOR VARIOUS SUBMARINE OUTFALLS

in physical dilution but all at a greatly reduced rate. Eq. 22 and the curves of Fig. 6, from which it was derived, suggest that there is a limit to the absolute potential for the receiving waters in absorbing pollution. The formula shows this in the rapid tapering off of the dilution ratio curve. The curves of Fig. 6 reflect that increasing initial dilution always results in decreasing the dilution ratio, showing that overall dilution, bacterially-measured or chemically-measured, is not increased in the same degree as the initial dilution.

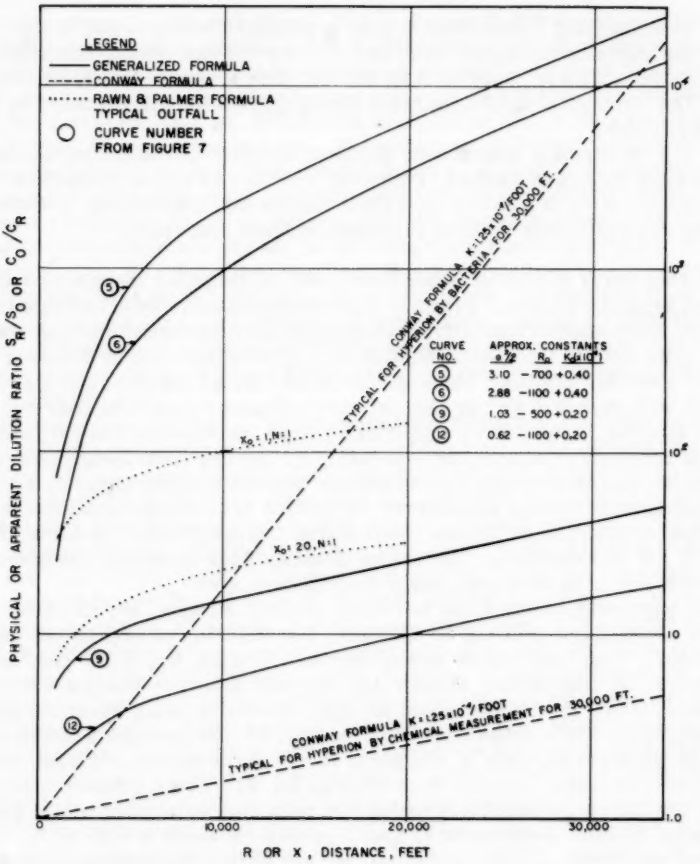
LEGEND
GENERALIZED FORMULA
CONWAY FORMULA
RAWN & PALMER FORMULA TYPICAL OUTFALL
CURVE NUMBER FROM FIGURE 7
PHYSICAL OR APPARENT DILUTION RATIO S_R/S_O OR C_O/C_R
R OR X, DISTANCE, FEET
CONWAY FORMULA K=1.25x10⁻⁴/FOOT TYPICAL FOR HYPERION BY BACTERIA FOR 30,000 FT.
CONWAY FORMULA K=1.25x10⁻⁴/FOOT TYPICAL FOR HYPERION BY CHEMICAL MEASUREMENT FOR 30,000 FT.
CURVE NO. | APPROX. CONSTANTS θ/Z | R_a | K_d(10⁴)
5 | 3.10 | −700 | +0.40
6 | 2.88 | −1100 | +0.40
9 | 1.03 | −500 | +0.20
12 | 0.62 | −1100 | +0.20
X_o = 1, N=1
X_o = 20, N=1
0
10,000
20,000
30,000
10⁴
10³
10²
10
1.0

FIG. 7.—GENERALIZED FORMULA FOR SPREAD AND DISSIPATION OF A SEWAGE FIELD IN SEA WATER

Descriptive power function curves of dilution ratio versus distance were developed for several of the outfalls studied. These took the form

$$\frac{S_R}{S_O} = K R^n \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

in which n, the exponent was found to vary from 0.11 to 0.37 for chemically-measured physical dilution and from 1.33 to 2.63 for "apparent," bacterially-measured dilution. However, ogee curves were generally found in the middle of the straight lines on the log-log plots, indicating the data were not entirely in accord with the presumed power function.

In summary, the investigations into the applicability of the various formulas for dilution and dissipation of sewage effluent injected into the ocean showed the following:

1. In estimating initial dilution over a proposed ocean outfall outlet or diffuser the three methods of Rawn and Palmer curves; Rawn, Bowermen, and Brooks "jet analysis" curves; and current flux formula, Eq. 4, should all be used. The average of their somewhat divergent results will probably be a satisfactory guide.
2. For describing subsequent physical dilution and bacterial decline, the formulas of Rawn and Palmer, Pomeroy, and Brooks and of Conway as herein modified, were each found to have their merits and limitations. Further discussion and evaluation of these formulas appears elsewhere.[14]

Decline Rates of Coliform Bacteria.—The anticipated discharge from the effluent outfall is a blend of primary effluent and standard-rate activated sludge effluent. When unchlorinated Hyperion plant effluent has been discharged through the 1-mile outfall into Santa Monica bay, a reduction rate of 90% each 6.5 hr for the existing modified high-rate activated sludge effluent, and a reduction rate of 90% each 3.8 hr for the primary effluent alone have been observed. These figures represent an empirical overall decline rate for coliform bacteria in the surface waters, incorporating all factors contributing to bacterial decline such as bactericide, sedimentation, dilution, and predation by zooplankton. Comparable figures established for decline of coliforms from the primary effluents discharged at Whites Point and at Orange County coastline are 4 hr and 1.5 hr, respectively. The values developed for Hyperion discharge were therefore felt to be both reasonable and conservative.

The engineers utilized the services of N. H. Brooks, M., ASCE as a consultant on matters relating to diffusers and diffusion of sewage effluent into sea water. They also made use of Brooks' formula, Eq. 9, for physical dispersion of an established sewage field by eddy diffusion within a water mass in motion having first ascertained its applicability by using specific field data involving chemically-measured dilutions. Eq. 19 incorporates a factor for decline of coliform bacteria in sea water by all of the agencies except turbulent diffusion. The latter factor is described by Eq. 11. The engineers established values for the corresponding time for 90% reduction as applied to the Hyperion discharge offshore and found it to have a geometric mean of 4.02 hr for one set of three observations, 8.91 hr for another set of five observations, and 2.53 hr for a third set of four observations. The overall geometric mean yielded a value of 4.72 hr. The engineers adopted a design value of 8 hr per 90% coliform reduction, as a matter of conservative practice, this value excluding only the effect of physical dilution by turbulent diffusion in causing coliform decline. This would mean, for example that if the coliform concentrations within a hypothetical sewage field were to be reduced from say 10,000 per ml to 10 per ml by the time the field moved from its origin to the surf area, a period of 24 hr would be required, ignoring any actual dilution benefit derived from turbulent diffusion.

In order to estimate the coliform densities of the unchlorinated Hyperion primary effluent scheduled to be discharged through the effluent outfall, a statistical analysis was made of the analytical data resulting from daily grab samples over a 1-yr period for the Hyperion effluent of the primary treatment units, the final plant effluent before chlorination, and the plant influent. Following the method of Pomeroy,[30] log probability plots of the results for the respective stages of treatment were made. Also arithmetic and geometric means were computed. The plots appear in Fig. 8. The most significant data appear in Table 4.

For many years, the experience of Los Angeles' Bureau of Sanitation has been that for the most probable number (MPN) of a series of beach counts at any of the beach stations in Santa Monica bay, a geometric value of 2.5 coliforms per ml was the approximate limit beyond which the Salt Water Bathing Standard would be exceeded, that is, 20% of the samples of the series would contain greater than 10 coliforms per ml. The engineers assumed that the coliform count of the future primary plant effluent would more nearly correspond to the present plant influent than the present effluent from the primary units. Thus, the geometric mean value of the plant effluent would be taken to be approximately 350,000 coliforms per ml. If this figure is combined with the required geometric mean of 2.5 for the beach stations, the required reduction in count will be in the ratio of 140,000 to 1. However, it was felt that the great length of the proposed outfall and the large dilute effluent field to be established by the type of diffusion proposed would tend to decrease the variations in the beach counts and increase the critical figure of 2.5. This would, in turn, reduce the ratio of 140,000 to 1.

For the purpose of the theoretical studies for diffusion and length of outfall, an initial coliform density of 600,000 coliforms per ml in the effluent discharge was used for all computations. This corresponds approximately to the primary effluent 50% occurrence value plus one standard deviation, and it is only slightly lower than these corresponding values for plant influent. This 600,000 per ml figure was felt to be conservative in view of the analytical data of Table 4 and the long term geometric mean of 400,000 coliforms per ml predicted by the Bureau of Sanitation. To reduce the 600,000 per ml effluent coliform density to not more than 10 per ml, the 80 percentile figure of the California Salt Water Bathing Standard, an overall reduction of at least 60,000 is required.

Prediction of Initial and Subsequent Dilution.—Considerations of outfall length during the conceptual stages of the design had always involved figures of at least 5 miles.[7] Bond issue funding itself had been based on this length. The approximate 5-mile length was developed on the basis of reaching deep water sufficient for provision of cold diluting bottom water for subsurface field stratification, and because of the limitation placed on economical submarine construction beyond that point due to skyrocketing diving costs. This proposed outfall length also harmonized reasonably well with water pollution abatement experiences alluded to earlier.

It was subsequently found that not only was this distance adequate for obtaining necessary initial dilution and subsequent bacterial decline, but also a relatively flat bottom area was available at a distance of about five miles directly seaward from Hyperion. As pointed out by Brooks,[29] this condition of

30 "Statistical Treatment of Bacterial Data," by Richard D. Pomeroy, Sewage and Industrial Wastes, Vol. 27, p. 85, January, 1955.

TABLE 4.—ONE YEAR ANALYSIS OF HYPERION PLANT FLOW, COLIFORM ORGANISMS PER MILLILITER[a]

Item (1)	Plant Influent (2)	Primary Effluent[b] (3)	Secondary Effluent (4)
Arithmetic Mean	515,000	452,000	222,000
Geometric Mean	366,000	259,000	113,000
Median	240,000	240,000	70,000
N_{50}(50% Occurrence)	350,000	250,000	100,000
N_{80}(80% Occurrence)	665,000	550,000	290,000
Standard Deviation			
Below N_{50}	160,000	95,000	31,000
Above N_{50}	700,000	650,000	377,000

[a] Data correspond to those plotted in Fig. 8.
[b] Used for outfall design.

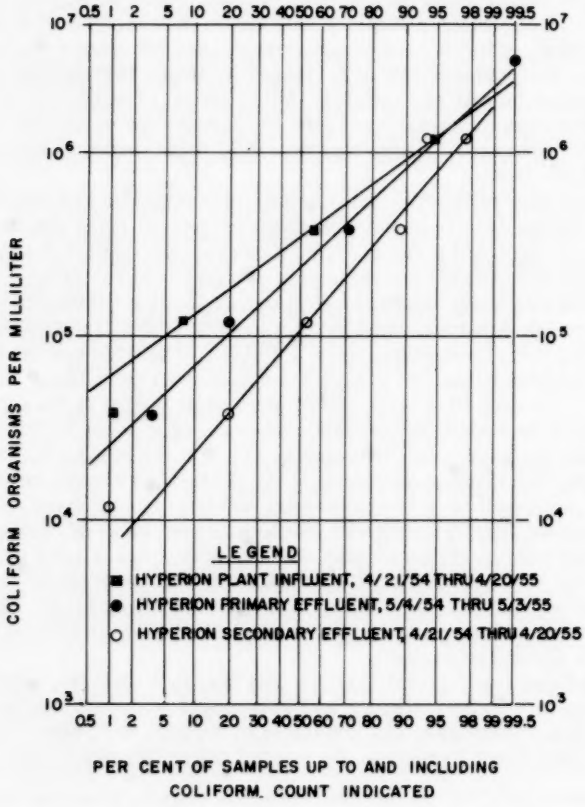
0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.5
10^7
10^6
10^5
10^4
10^3
COLIFORM ORGANISMS PER MILLILITER
LEGEND
HYPERION PLANT INFLUENT, 4/21/54 THRU 4/20/55
HYPERION PRIMARY EFFLUENT, 5/4/54 THRU 5/3/55
HYPERION SECONDARY EFFLUENT, 4/21/54 THRU 4/20/55
PER CENT OF SAMPLES UP TO AND INCLUDING COLIFORM COUNT INDICATED

FIG. 8.—PROBABILITY PLOTS OF HYPERION TREATMENT PLANT COLIFORM DATA

levelness simplifies manifold design and predictability because under all conditions of discharge, the discharge from an individual diffuser port remains directly proportional to the total discharge of the diffuser. This factor was, however, of secondary importance in determining the ultimate location of the outfall diffuser.

The concept of the anticipated magnitude of initial dilution for the proposed outfall-diffuser system was developed approximately as follows:

Minimum flow, any time	100 mgd	(155 cfs)
Design flow, year 2000 avg	420 mgd	(650 cfs)
Design flow, year 2000 peak	600 mgd	(928 cfs)

Preferred type of diffuser—two legged wye type

Length of individual diffuser leg—4,000 ft, considered practical maximum, this application

Peak design flow per foot of diffuser length—0.116 cfs

Approximate outlet port spacing to prevent mutual interference at 200 ft diffuser depth—about 100 feet, between ports on same side of pipe assuming ports "staggered" on alternating sides of pipe

Total number of diffuser ports, both legs, both sides of pipe—Approximately 160

Port discharge at design peak flow—5.8 cfs, average

Port discharge at minimum flow—0.6 cfs approximate average

Cross sectional area of ocean outfall also of two main diffuser legs—113.1 sq ft.

Aggregate diffuser port area—about 56 sq ft (Approximately one-half of cross section area of pipe)

Average port discharge velocity at design peak flow—16.5 fps

Average port discharge velocity at minimum flow—2.77 fps

Average port diameter at 0.35 sq ft per average diffuser port—8 in. (0.67 ft)

Average Froude Number (Should be > 1 to assure port flowing full)

$$F = \frac{v}{\sqrt{g' D}} = \frac{v \text{ (fps)}}{\sqrt{(0.027)(32.2)(0.67)}} = \frac{v}{0.761} \quad \ldots\ldots\ldots \quad (24)$$

At minimum flow it was 3.64

At design peak flow it was 21.7

Average ratio of port ocean submergence to port diameter, Y/D 195-0.67 = 290^{26}

Initial Dilution at Minimum flow was 60, minimum and at design peak flow was 60, Minimum

The foregoing arrives at a predicted minimum initial dilution presuming insignificant ocean currents. In this case, the individual jets create a movement of diluting water past the diffuser legs. This quiescence is rarely, if ever, the case. A 0.1 knot current (0.169 fps) intersected by, say, a 4,000 ft width of diffuser pattern in 200 ft of water depth is capable of supplying 135,000 cfs diluting flow past the diffuser. Because of upward deflection of the injected effluent, not all of this moving current becomes effective in diluting the rising jets. However, if only 45% of the moving mass of water is engaged in diluting the injected effluent, then the peak discharge of 928 cfs would receive an initial dilution of

$$\frac{135{,}000\ (0.45)}{928 = 65.5}$$

Stated otherwise, the initial effluent field resulting from initial submarine injection would be 4000 ft wide and 90 ft deep moving at an average velocity of 0.169 fps, equivalent to 60,900 cfs of effluent field as compared to 928 cfs of effluent or an initial dilution of 65.5, for this particular current orientation and field width.

Subsequent dilution of the effluent field formed initially by the rising discharge from the diffusers as formulated by Brooks contains two basic elements. The first, Eq. 9, relates purely to physical dilution due to turbulent diffusion, and the second, Eq. 12, represents all of the agencies responsible for decline of sewage bacteria in the sea water except the agency of physical dilution due to turbulent diffusion. The overall reduction of sewage bacteria can be computed by combining these two basic elements, Eq. 19. The coefficient of eddy diffusivity, entering into the physical dilution component, is capable of evaluation by field measurement. This component is developed rationally. The coefficient of bacterial decline is similar to that originally developed by H. Chick,[31] being an empirical exponent describing the behavior of the bacterial population over a period of time. This exponent must also be evaluated such as by dye patch sampling and coliform determinations at intervals of time. The advantages of the Brooks Formula are that the elements influencing subsequent effluent field dilution such as ocean turbulence, initial field width, and ocean currents can be individually taken into consideration to suit the actual or anticipated conditions of operation, or both, with any diffuser system and receiving waters. The bacterial decline exponent, however, can be computed by the formula from field measurements and applied to the formula to predict overall bacterial decline. The physical dilution component permits extrapolation. This then provides a somewhat more refined means of expressing and anticipating overall bacterial decline within the sewage field than does a single bacterial decline exponent incorporating all the agencies of decline including dilution by turbulent diffusion. In some circumstances the additional precision to be obtained by use of the Brooks formula is completely warranted. In other cases, especially where the value of "time" is large and ocean turbulence is relatively low, the bacterial decline component predominates, and the unavoidable uncertainty of the bacterial decline coefficient obscures the precision of the physical dilution component.

The Brooks formula, Eq. 19, in basic form is

$$\frac{S_t}{S_o} = \frac{C_o}{C_t} = \frac{e^{kt}}{\operatorname{erf}\dfrac{b}{4\sqrt{\epsilon t}}} = \text{Dilution Ratio} \quad \ldots\ldots\ldots (25a)$$

$$= \frac{e^{kt}}{\operatorname{erf}\dfrac{b}{4\sqrt{\epsilon_o \dfrac{b^2}{24\,\epsilon_o}\left[\left(\dfrac{8\,ct}{b^{2/3}} + 1\right)^3 - 1\right]}}} \quad \ldots\ldots\ldots (25b)$$

31 "Investigation of the Laws of Disinfection," by Harriet Chick, Journal of Hygiene, Vol. 8, p. 655, 1908.

$$= \frac{10^{\frac{t}{T}}}{\operatorname{erf}\frac{1.23}{\sqrt{\left(\frac{8C_1 t}{b^{2/3}} + 1\right)^3 - 1}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (25c)$$

in which e is the base of natural logarithms; k denotes the decline rate of sewage bacteria in sea water due to all causes except turbulent diffusion; T is T-90,

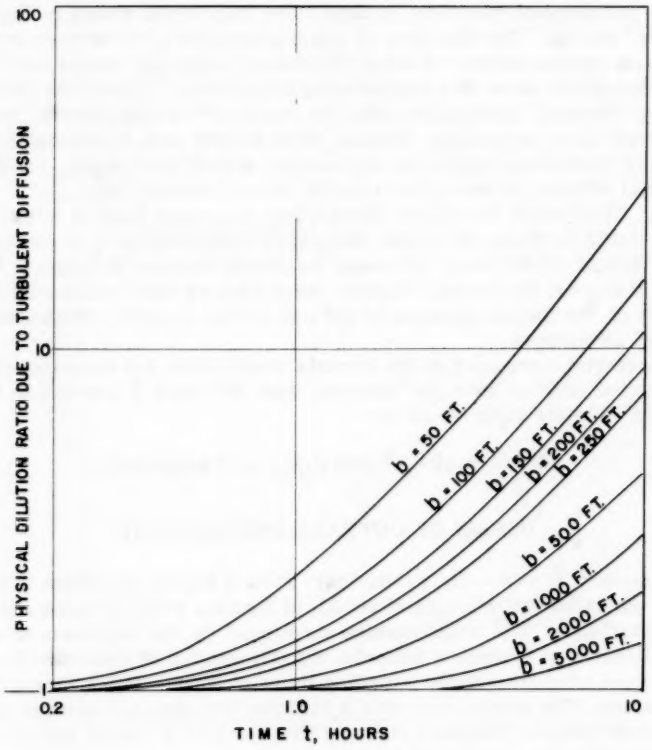

FIG. 9.—PHYSICAL DILUTION OF A SEWAGE FIELD IN OCEAN WATER OF MILD TURBULENCE COMPUTED FROM BROOKS FORMULA

the equivalent of k on a scale of common logarithms, the time hours, for 90% decline of sewage bacteria in sea water due to all causes except turbulent diffusion; ϵ refers to the turbulent diffusion coefficient or eddy diffusivity; ϵ_o is the initial eddy diffusivity, Santa Monica bay taken as 1.84 $b^{4/3}$ ft $^{2-3}$ per hr; C_1 is 1.84 ft $^{2/3}$ per hr, a constant, applicable to the mildly turbulent Santa Monica Bay; C_1 is 3.68 ft $^{2/3}$ per hr, applicable to ocean receiving water of

"normal" turbulence; and C_1 is 7.36 ft. 2/3 per hr, presumed generally applicable to ocean waters of "greater than normal" turbulence.

The dilution ratio, S_t/S_o, is the reciprocal of the concentration ratio and is taken to represent overall "apparent" bacterial decline if both the numerator, $10^{\frac{t}{T}}$, and the denominator, erf, and so on, are used. If only the denominator is used and the numerator is equal to one, then the dilution ratio represents the increase in physical dilution alone, over that initially present, as a function of time, initial field width, and coefficient of eddy diffusivity. Fig. 9 presents plots of physical dilution ratio for an effluent field moving away from a submarine diffuser under the influence of a uniform unidirectional current so that the only independent variable is time. For the curves shown in Fig. 9 C_1 is 1.84 ft $^{2/3}$ per hr. The families of curves presented for various initial field widths are representative of mildly turbulent receiving waters such as Santa Monica Bay whose value was measured by Gunnerson.[12] Use of the "Universal" value for "normal" turbulence cited by Pearson[24] would steepen the curves. The curves show increasing dilution ratio benefit with increasing time, with increasing turbulence, and with decreasing initial field depth. Nevertheless high initial dilution is desirable from the overall standpoint.

As an illustration for Santa Monica Bay, a sewage field of initial width of 5,000 ft would increase its initial dilution by only 1.35 times in 10 hr, but one of 50 ft initial width would increase its initial dilution 38 times in the same period. However, the overall dilution of the former field would still be better than that of the former because of its high initial dilution, which also affords additional advantages.

The bacterial component of the formula would state, for example, that if the non-diffusion decline rate for bacteria was 90% each 8 hr, then in 24 hr the "apparent" dilution ratio would be

$$10^{24/8} = 10^3 = 1000 \text{ times or 3 magnitudes}$$

DESIGN OF OUTFALL AND DIFFUSER

Preliminary Trials.—The preliminary work of Pomeroy and that of the engineers[32] indicated that a length of outfall of 5 miles would be adequate to meet all requirements. The relationships developed by the engineers were based on a utilization of Conway's formula, Eq. 17, applied to field data for a number of ocean effluent outfalls including those of Whites Point, Orange County, and Hyperion. The preliminary trials utilizing this formula and the results of these investigations indicated strongly that 5 miles of outfall length was adequate; the chief uncertainty lay in selecting the precise empirical coefficients and subsequently establishing the predicted beach counts. The offshore bacterial and oceanographic data provided by the Hancock Foundation and the Bureau of Sanitation during the course of the preliminary design and first stages of the final design formed the basis of final design of the ocean outfall and diffuser. Selection of outfall diameter was based on economic studies of a more routine nature, in which fixed and operating costs of the outfall-diffuser system and pumping plant were investigated, for a number of possible combinations.

32 Preliminary Report of Ocean Outfall Investigation for Hyperion Treatment Plant, City of Los Angeles, Hyperion Engineers, September 30, 1955.

Alternate Diffuser Schemes.—The four alternate diffuser schemes depicted in Fig. 10 were considered in some detail prior to final design. For Fig. 10 the effect of lateral field spread by rising jets and diffusion is considered inversely proportional to current velocity. These were all adaptable to the bottom topography and other physical considerations encountered and were initially laid out to yield approximately comparable results for roughly equivalent total costs of outfall and diffuser. They were developed to permit evaluation of certain basic patterns from considerations of performance primarily and of cost only

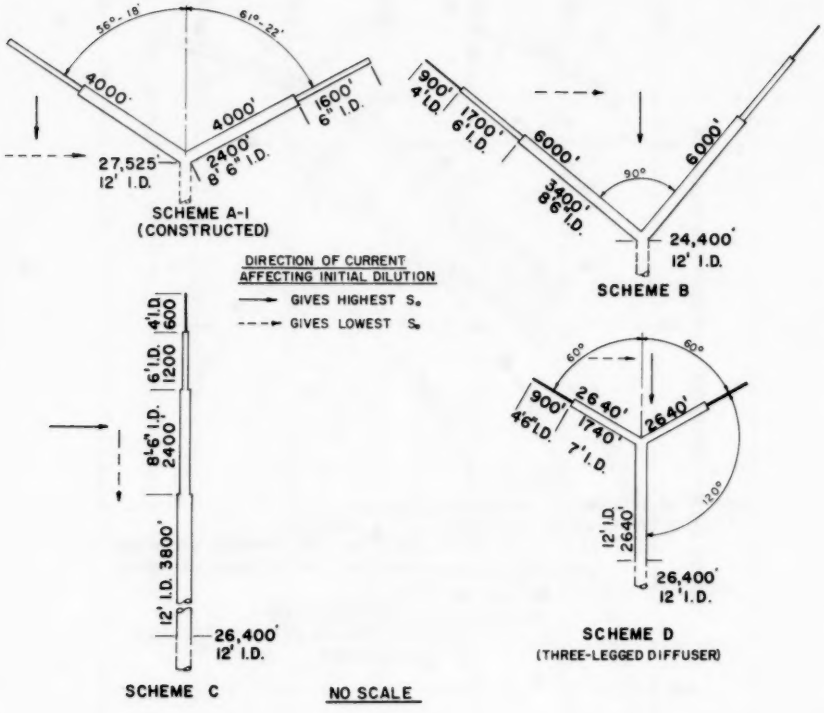

FIG. 10.—ALTERNATE DIFFUSER-OUTFALL SCHEMES STUDIED

secondarily. Performance aspects included initial dilution, and subsequent bacterial decline for various directions and magnitudes of ocean current, preliminary predictions of manifold and pumping hydraulics, ease of maintenance, and amenability to future extension if required. Scheme A-1 was finally selected, designed, and constructed.

Influence of Currents Upon Dilution and Coliform Decline.—Figs. 11 and 12 show the predicted coliform densities at the surf zone for alternate diffuser schemes A-1 and C under the influence of assumed uniform currents of various on-shore directions and various magnitudes. The curves for schemes B and

D were remarkably similar to those of Scheme A-1. A coliform density of 600,000 coliforms per ml in the discharge effluent is presumed and the minimum dilution anticipated from any diffuser under any condition of current and discharge is taken to be 60. All dilutions are based on a peak design flow of 600 MGD. Stronger currents give higher initial dilution but move the field towards the surf zone more rapidly than weak currents, allowing decreased time for bacterial decline.

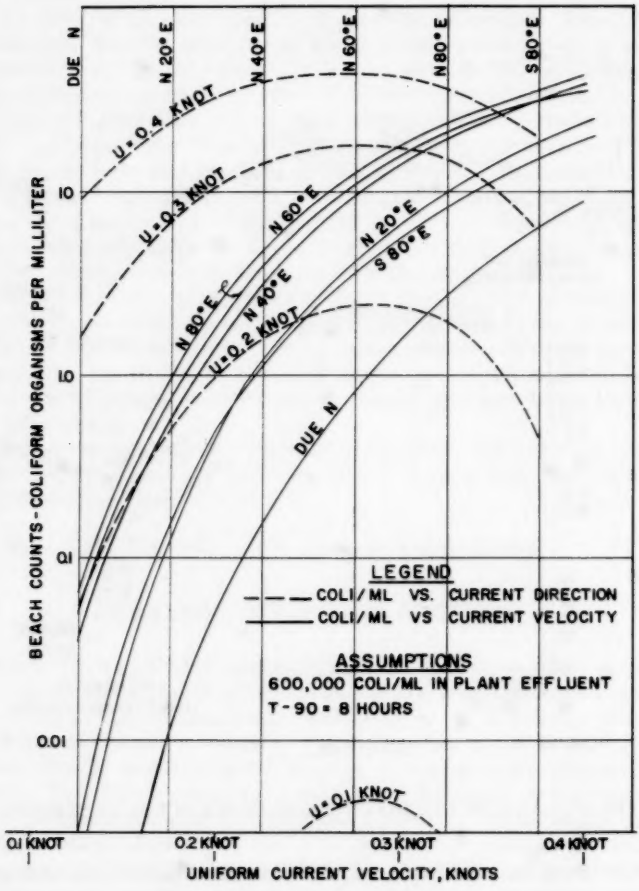

FIG. 11.—BEACH COUNTS VERSUS OCEAN CURRENTS FOR ALTERNATE SCHEME A-I

The so-called "Current Flux Formula," Eq. 4, that was used in a previous illustration of initial dilution, was utilized in the computations. In the computations on which the diffuser operation figures are based, the depth of effluent field was assumed to be 90 ft. This figure was obtained in the case of the Scheme

A-1 diffuser by computing the depth required to obtain an initial of 60 under the action of a 0.1 knot shoreward current, a discharge of 600 MGD, and a minimum field width of about 3700 ft, the minimum estimated diffuser projection for any current heading towards critical shore areas. Actually, an initial dilution of 60 can be reasonably anticipated under a condition of no current. The

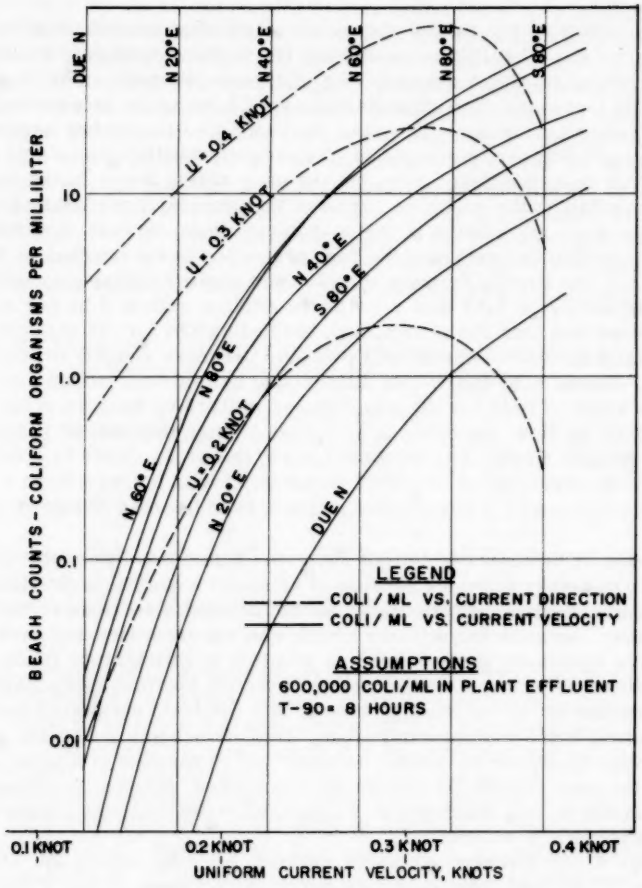

FIG. 12.—BEACH COUNTS VERSUS OCEAN CURRENTS FOR ALTERNATE SCHEME C

widths of the sewage field used for the computations of wye diffusers were the widths of water masses intercepted under the various current directions with an allowance for additional spread of the forming field due to diffusion and the paths of the rising jets. In the case of the straight line diffuser, scheme C, the minimum width of the field was that which, when combined with a depth of

90 ft and a minimum 0.1 knot onshore current parallel to the diffuser axis, yielded an initial dilution of 6.0. For 600 MGD this width would be about 3700 ft. Whenever the 0.1 knot current was at an oblique angle to the outfall sufficient to result in a width greater than that computed for onshore current, the assumed depth was decreased to maintain the dilution at 60. However, with a normal current presentation the initial dilution would exceed 60 by an appreciable amount.

A comparison of the curves showed no outstanding advantage of one type of diffuser over the other. The wye diffuser (A-1) showed slightly better results for the critical on-shore currents, the difference becoming more apparent as the velocity increases. The straight diffuser (C), as might be expected, showed slightly better advantage with cross current. The theory has been advanced that a straight diffuser intercepts random currents better than a wye type diffuser. This is undoubtedly true, but the over-riding factor in the case of the Hyperion outfall is the on-shore currents that generally predominate and may potentially cause more beach pollution than any other current. For this reason the wye type diffuser provides the best protection to the beaches at Hyperion. In all cases the Brooks formula, Eq. 26, was applied utilizing a coefficient of initial diffusivity of 1.83 and a coliform decline rate of 8 hr per magnitude.

For flows less than the design flow, the field width for the straight diffuser with an axial on-shore current will probably decrease roughly in proportion to the flow, maintaining the initial dilution due to aspirator action alone. However, the width of field for the wye diffuser is fixed by the size of the diffuser irrespective of flow, resulting in an initial dilution that will be larger for the smaller sewage flows. For the near future, therefore, there is a distinct advantage in the wye-type of diffuser. In addition, with the wye there is a better possibility of forming a submerged effluent field because of higher initial dilution.

Selection of Diffuser and Outfall Pattern.—Scheme A-1 was selected in preference to the other schemes because it provided a desirable physical dilution and coliform decline pattern under all the critical directions of current past the diffuser, because the diffuser length was not excessive and gave promise of reliable hydraulic performance in addition to satisfactory initial dilution, and because the scheme was amenable to outfall cleaning and possible future outfall extension. In addition to this it had the least estimated construction cost of any of the combinations studied. Neither scheme C nor scheme D was particularly amenable to outfall extension or to passage of a cleaning device through the main outfall for maintenance purposes. Scheme B, although similar in concept to thst of scheme A-1 appeared more costly and less reliable in overall hydraulic performances.

In Fig. 4, the average on-shore current velocity within the 110° sector shown is 31 fpm, and these on-shore currents represent 45% to 50% of the total experience of currents passing the 5 miles offshore point. No direction predominated more than 6% of the time.

From a statistical analysis of these off-shore current data, Brooks deduced that the travel time from 5 miles off Hyperion to any particular shore station would exceed 15 hr in 98% of the time. The Bureau of Sanitation computed the probable fastest movement of the effluent field from the 5-mile point as reaching the surf zone after a minimum of some 20 hr. The Hancock Foundation observed no current velocity past this 5-mile point in excess of 0.50 knot and determined that outside of the littoral waters the usual velocities were 0.27 knot

or less. These values would not be expected to be exceeded as much as 20% of the time.

Figs. 11 and 12 are based on an assumption that the effluent field travels on shore in a straight line under influence of the uniform current velocity throughout the entire distance. This assumption over-simplifies the actual motions in the ocean, but the assumption is on the side of safety. Both the direction and the magnitude of off-shore currents have been observed to vary considerably so that an effluent field actually never moves on-shore along a straight line but rather along a curving path thereby increasing the actual transit time. Also, the currents are found to decrease in magnitude as the water mass approaches the shallower water. Table 5 illustrates the computations for the adverse current direction of north 90° east as depicted in Fig. 11. In all the diffuser calculations, the coliform density of the discharge effluent was taken at 600,000 coliforms per ml, and the rate of bacterial decline was 90% each 8 hr due to all causes except dilution by turbulent diffusion. Table 5 illustrates that for increasing magnitudes of onshore current passing the diffuser; the initial dilution is increased, due to the greater supply of diluting water available, but the time for coliform decline is curtailed and the overall coliform decline becomes

TABLE 5.—SAMPLE COMPUTATION OF ADVERSE CURRENT DIRECTION

Current Velocity knot	Initial Dilution, S_o	Dilution Ratio $\frac{S_R}{S_o} = \frac{C_o}{C_t}$			Beach Count coliforms per ml
		physical dilution	bacterial decline	overall decline	
(1)	(2)	(3)	(4)	(5)	(6)
0.10	115	3.34	4.75×10^5	15.8×10^5	0.0033
0.20	225	1.88	692	1300	2.05
0.30	335	1.48	77.7	115	15.6
0.40	445	1.29	28.1	36.2	37.2

drastically reduced with increased current velocity. Thus, the 0.30 knot onshore current results in a coliform count of 15.6 at the beach and corresponds to a travel time of 15.1 hr. A travel time of 22.7 hr corresponds to a current magnitude of 0.20 knot and a beach count of 2.05 coliforms per ml, well within the requirements of the State Water Pollution Control Board.

The onshore current magnitude estimated by the Hancock Foundation to exceed 0.27 knot less than 20% of the time is found by interpolation to reach a peak at about north 60° east current direction in Fig. 11 and to correspond to a beach count of about 10 coliforms per ml. The measured offshore coliform decline rates previously described showed an overall geometric mean of 4.72 hr per magnitude exclusive of dilution due to turbulent diffusion as compared to the value of 8 hr used in preparation of Figs. 11 and 12. Therefore, both the conservative allowance for coliform decline rate adopted in the design computations and the statistical probabilities showing a low frequency of occur-

rance of adverse currents combine to indicate strongly a future compliance of the 5-mile outfall within prescribed standards.

SUMMARY

1. Extensive investigations were made prior to and concurrent with the design of the 5-mile ocean outfall for effluent disposal for discharge of unchlorinated plant effluent from Hyperion into Santa Monica Bay. These investigations included specific studies for the proposed discharge and also general studies on the nature of ocean disposal of sewage effluent and actual performance of a number of major outfalls.
2. The general studies confirmed the methods utilized in the design of the 5-mile outfall.
3. The general and specific studies demonstrated the variability of physical, chemical, and bacteriological data that must be anticipated in actual outfall performance, but they also exhibited definite trends among the data usually capable of mathematical formulation.
4. The specific studies for the proposed discharge indicated current patterns and coliform decline rates adequate to provide satisfactory physical dilution of the discharged effluent and reduction of bacterial concentration to meet prescribed standards.
5. A vast amount of effort and expense is necessary to obtain significant field data on oceanography, and bacteriology requisite for a proper design of a major ocean outfall such as the 5-mile Hyperion outfall. The data that have been obtained for this outfall, although not complete, are considered sufficient under the circumstances.
6. Operation of the 5-mile Hyperion outfall has been in progress as of March, 1961 for approximately one year, and results have generally been satisfactory. Evaluation of the sanitary design of the outfall will become possible as flows increase and as more and more performance data become available.

ACKNOWLEDGMENTS

The writer is pleased to acknowledge the assistance, cooperation, and participation of the many persons who contributed to the investigations and sanitary design of the 5-mile Hyperion effluent outfall, especially N. B. Hume, F. ASCE, R. D. Bargman, F. ASCE, W. F. Garber, F. ASCE, and C. G. Gunnerson, F. ASCE of the city of Los Angeles Bureau of Sanitation; N. H. Brooks, M. ASCE of the California Institute of Technology; R. E. Stevenson of the Allan Hancock Foundation; F. R. Bowerman, M. ASCE, of the Los Angeles County Sanitation Districts; and R. R. Alvy, F. ASCE, D. R. Miller, M. ASCE, E. H. Graham, Jr., M. ASCE, and D. L. Narver, Jr., F. ASCE of Hyperion Engineers. The consulting engineers coordinated as the Hyperion Engineers consisted of Holmes and Narver; Daniel, Mann, Johnson, and Mendenhall; and Koebig and Koebig.

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TRICKLING FILTRATION DESIGN AND PERFORMANCE

By W. Wesley Eckenfelder, Jr.,[1] M. ASCE

SYNOPSIS

During the past two decades various formulations have been developed to describe trickling filtration performance. Most of these have been based on empirical correlations of filtration data. More recent studies have been directed toward defining trickling filtration hydraulics. From these various investigations filtration design formulations have been developed relating depth, hydraulic loading, and media characteristics. This paper attempts to statistically correlate filter performance to the aforementioned variables and to develop generalized design formulations for domestic sewage treatment based on these analyses.

MECHANISM OF BOD REMOVAL IN TRICKLING FILTERS

Notation.—The letter symbols adopted for use in this paper are defined where they first appear and are arranged alphabetically, for convenience of reference, in Appendix I.

Removal of BOD by trickling filters results from biosorption and coagulation of that portion of the flow that passes rapidly through the filter. Progressive removal of the more soluble constituents occurs from that portion of the flow with increased residence time on the filter slimes. As in the activated sludge process BOD removal can be expected to be proportional to both the mass of active filter slime and to the time of contact of the waste in the filter. The mass of active film is related to the surface area of filter slime and to the depth of aerobic activity. The time of contact is primarily related

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SA 4, July, 1961.

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to the hydraulic loading. At the low BOD concentrations usually applied to a filter and assuming first order kinetics these variables may be related:

$$\frac{L_e}{L_o} = e^{-kt} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which L_e is the BOD remaining in the filter effluent; L_o represents the BOD applied to the filter; k is a coefficient incorporating the surface area of active film per unit volume; and t denotes time of contact.

For sewage and complex industrial wastes, Eq. 1 may require modification to a retardant function, because the various waste constituents differ in the ease with which they are removed. This will be discussed in more detail in a subsequent section of this paper.

The residence time, t, in Eq. 1 has been related to the depth and hydraulic loading of the filter and to the physical characteristics of the filter media (1), (2), (3), (4).[2]

$$t = \frac{C\,D}{Q^n} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The constant, C, and the exponent, n, will vary with the type of filter media and the hydraulic characteristics (4). The exponent has been shown to approach 1/3 for turbulent flow and 2/3 for laminar flow (5). J. H. McDermott, A.M. ASCE (6), determined an exponent of 0.55 for synthetic sewage passing over a 23 ft column of $3\frac{1}{2}$ in. balls. K. L. Shulze (7) found an exponent of 0.66 for flow over a screen filter. A study of residence time in Dowpac plastic media by E. H. Bryan and D. H. Moeller (8) showed n to be 0.5. Residence time is increased by the presence of slime and capillary water storage (5). Because BOD removal is proportional to time of contact it has been related to hydraulic loading by the relationship (7).

$$\frac{L_e}{L_o} = C\,Q^n \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The exponent n has been determined for several biological filtration systems using Eq. 3. These results are summarized in Table 1.

It was implied in Eq. 1 that BOD removal is related to the surface area of active film per unit volume of filter media. This is analogous to the concentration of mixed liquor solids in the activated sludge process. The surface area of film frequently varies with the type and distribution of filter media. It has also been shown to vary with the filter depth and season in many cases (15), (16). This can be mathematically approximated:

$$C \sim \frac{1}{D^m} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which C is related to the mean active filter film per unit volume throughout the filter depth. Eq. 1 can be modified to include the residence time and the density of film by combining with Eqs. 2 and 4.

$$\frac{L_e}{L_o} = e^{(-K)(1/D^m)(D/Q^n)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5a)$$

[2] Numerals in parentheses refer to corresponding references in the Appendix II.

and combining terms

$$\frac{L_e}{L_o} = e^{(-K\,D^{1-m})/Q^n} \quad \ldots\ldots\ldots\ldots\ldots (5b)$$

The filter film was approximately uniformly distributed in studies conducted by McDermott in a 23-ft column of $3\frac{1}{2}$ in. balls and by Shulze on a screen filter. In these cases the exponent m in Eq. 4 becomes zero and Eq. 5b reduces to:

$$\frac{L_e}{L_o} = e^{(-K\,D)/Q^n} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5c)$$

McDermott's results are shown in Fig. 1 in which the exponent n on hydraulic loading was determined as 0.67.

In many cases the film distribution, activity, and composition vary with the filter depth, and the exponent m in Eq. 4 is greater than zero. In this case the exponent on depth in Eq. 5b is less than 1.0. The effect of depth for several filtration studies is shown in Fig. 2.

TABLE 1.—EFFECT OF HYDRAULIC LOADING ON BOD REMOVAL

Filter (1)	n (2)	Reference (3)
Rock filters, 3-4 ft deep	0.60	(9) (10) (11)
Rock filters, 6 ft deep	0.58	(12)
Rock filters	0.44	(13)
Screen filter	0.67	(7)
23 ft column, $3\frac{1}{2}$ in. balls	0.67	(6)
Rock filters	0.50	(14)
Rock filters	0.42	(19)
Rock filters	0.41	(21)

Eq. 5b presumes that all the components of the organic waste are removed at the same rate. There is considerable evidence, however, that in sewage and complex wastes the removal decreases with concentration or time, because the more readily assimilable components will be removed more rapidly. This requires that Eq. 5b be modified to a retardant form of equation to describe the overall removal. Such a form of equation was found by the writer to reasonable correlate published filter performance data.

$$\frac{L_e}{L_o} = \frac{100}{1 + \dfrac{C\,D^{(1-m)}}{Q^n}} \quad \ldots\ldots\ldots\ldots\ldots (6)$$

The constant C and the exponents (1-m) and n were determined by multiple regression analysis for several sets of filter performance data. These are summarized in Table 2.

The results of Ingram and Grantham showed the exponent on depth to be greater than 1.0 implying an increase in filter film per unit volume with increasing depth. In the case of Ingram's experiments, this may be attributed

to the sectionalized filter and positive aeration of the filter media. The performance data of various filtration experiments are plotted according to Eq. 6 in Fig. 3.

The variations in the filtration characteristics as shown in Table 2 are attributable to film and flow distribution, media characteristics, temperature variations, and other factors characteristic of the filter. In order to deter-

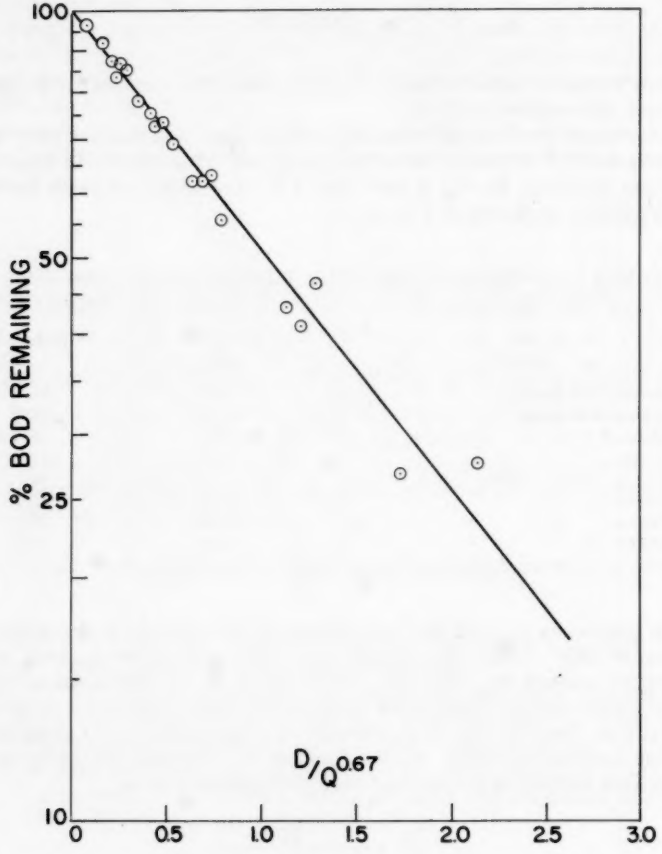

FIG. 1.—BOD REMOVAL CHARACTERISTICS OVER A 23 FOOT COLUMN OF $3\frac{1}{2}$ INCH BALLS (McDERMOTT (6))

mine a formulation applicable to the design of rock filters treating domestic sewage, mean values of the coefficients were determined resulting in the relationship:

$$\frac{L_e}{L_o} = \frac{100}{1 + (2.5)\frac{D^{0.67}}{Q^{0.50}}} \quad \ldots\ldots\ldots\ldots\ldots (7)$$

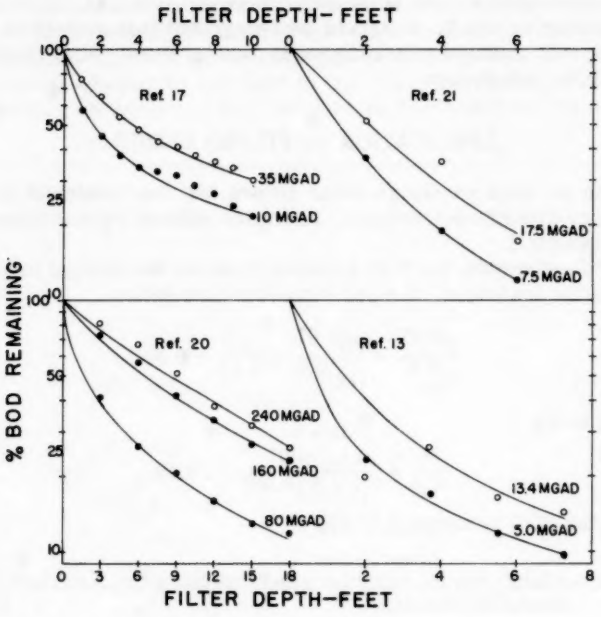

FIG. 2.—EFFECT OF DEPTH ON FILTRATION PERFORMANCE

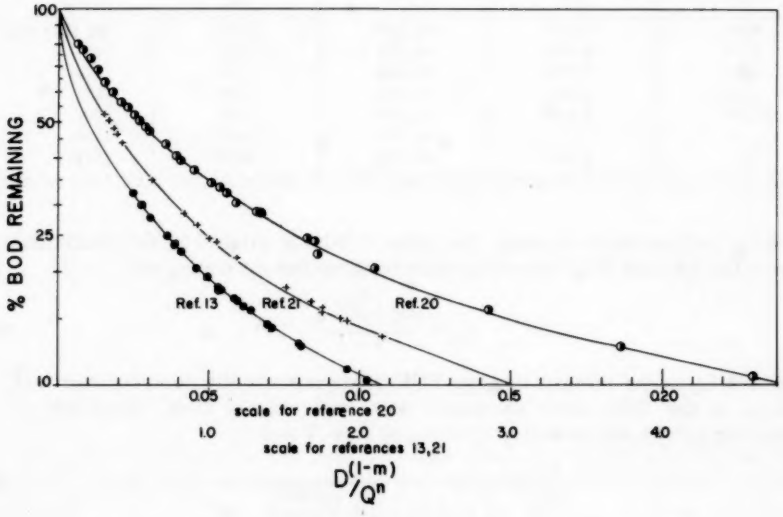

FIG. 3.—FILTRATION CHARACTERISTICS ACCORDING TO EQUATION 6 FOR VARIOUS PUBLISHED DATA

The filter performance data from references (9),(10),(11),(17) are shown in Fig. 4 according to Eq. 7. It should be recognized that deviations from this relationship will result from changes in temperature, filter media, waste characteristics, and so forth.

APPLICATION TO FILTER DESIGN

Eq. 7 can be used to design stone filters for the treatment of domestic sewage with or without recirculation. Filtration without recirculation is illustrated by Example 1.

Example 1.—Compute the BOD removal from settled sewage in a 6 ft filter with a hydraulic loading of 12 mgad without recirculation.

$$\frac{D^{0.67}}{Q^{0.5}} = \frac{(6)^{0.67}}{(12)^{0.5}} = \frac{3.3}{3.45} = 0.96$$

Substituting in Eq. 7

$$\frac{L_e}{L_o} = \frac{100}{1 + 2.5\ (0.96)} = 29.4\%$$

Therefore, the BOD removal is 70.6%.

TABLE 2.—TRICKLING FILTRATION PERFORMANCE CHARACTERISTICS - DOMESTIC SEWAGE

C (1)	(1-m) (2)	n (3)	Coefficient of correlation (4)	Reference (5)
4.23	0.365	-0.558	0.78	(9) (10) (11)
4.25	0.658	-0.411	0.87	(13)
0.29	0.774	-0.228	0.87	(17)
0.098	0.681	-0.175	0.84	(17)
37.77	1.204	-1.129	0.97	(20)
...	...	-0.782	0.87	(19)
3.0	1.49	-0.753	0.90	(21)

When recirculation is used, the influent BOD is diluted by the recirculated flow. The applied BOD including recirculation can be computed:

$$L_o = \frac{L_a + N\,L_e}{N + 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which L_a is the influent sewage BOD; N represents the recirculation ratio; and L_o is the BOD after admixture with recirculated flow. The filter performance can be estimated by combining Eqs. 7 and 8

$$\frac{L_e}{L_a} = \frac{1}{(1 + N)\left(1 + 2.5\,\frac{D^{0.67}}{Q^{0.5}}\right) - N} \quad \ldots\ldots\ldots\ldots (9)$$

It is recognized that the treatment characteristics of the recirculated flow will be less than that of the applied waste due to the retardant nature of the removal reaction. It would be necessary to modify Eq. 9 to consider this effect in a manner similar to that done in the NRC formulation. The application of Eq. 9 to filter design using recirculation is illustrated by Example 2.

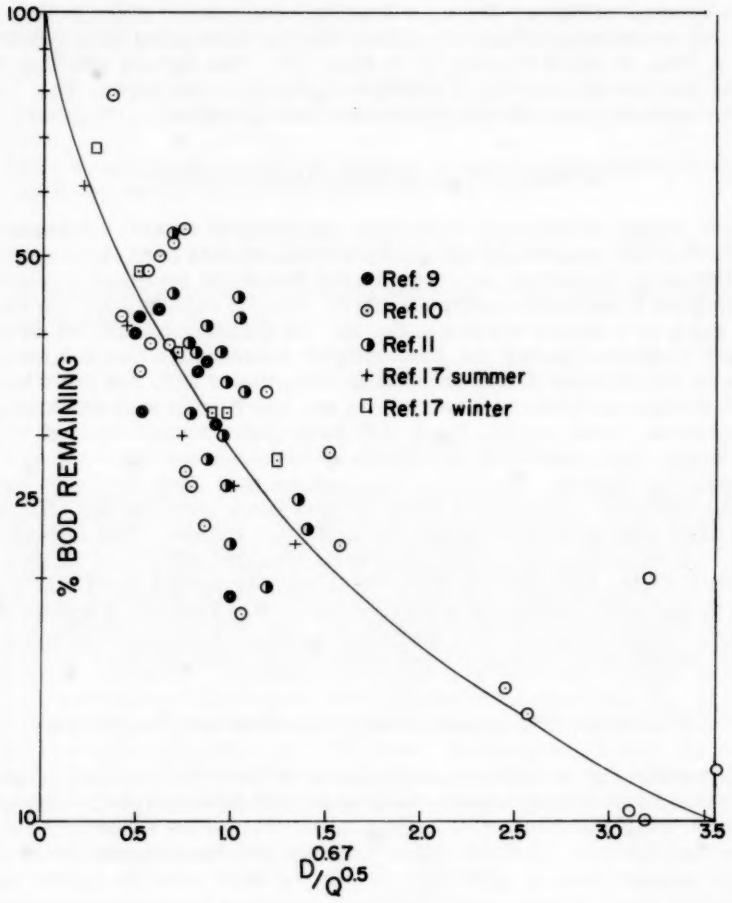

FIG. 4.—FILTRATION DESIGN CURVE

Example 2.—Compute the recirculation ratio to obtain 85% BOD removal from an initial BOD of 200 ppm using a depth of 4 ft and a hydraulic loading of 20 mgad.

$$\frac{D^{0.67}}{Q^{0.5}} = \frac{(4)^{0.67}}{(20)^{0.5}} = \frac{2.52}{4.48} = 0.56$$

From Eq. 9

$$\frac{L_e}{L_a} = 0.15 = \frac{1}{(1 + N)(1 + 2.5)(0.26) - N}$$

Solving for N

$$N = 3.0$$

It is recognized that a filter has a limiting loading above which further removal and assimilation cannot occur. This limiting loading has been discussed by C. J. Velz, F. ASCE (1), and V. T. Stack (18). The factors affecting this limiting load are developed in a subsequent section of this paper. Eqs. 7 and 9 are considered applicable only below the limiting loading.

OTHER FILTER DESIGN FORMULATIONS

Filter design formulations have been developed by several investigators. J. M. Fairall (14) statistically analyzed performance data from trickling filter installations in the western part of the United States and developed a relationship in terms of hydraulic loading and depth. Velz (1) related BOD removal to filter depth in a manner similar to Eq. 5c. His formulations did not directly consider hydraulic loading but differentiated between high rate and low rate filters in the constant K and the maximum fraction of BOD that could be removed through the filter. A limiting load was specified beyond which no further removal could occur. Stack (18) developed a modification of Velz's formulations that considered successive saturation of the filter bed with increased filter loading. Empirical formulations have been developed by the National Research Council (10) which related filter BOD removal efficiency to the filter loading (lb BOD applied per unit filter volume). This formulation considered the decreasing removability of recirculated BOD. R. S. Rankine (9) developed design formulations from the Upper Mississippi River Standards. These formulations considered a constant percentage removal of applied BOD including recirculation up to a maximum loading. A range of depth and hydraulic loading were specified over which the formulations were applicable.

OXYGEN TRANSFER AND TEMPERATURE RELATIONSHIPS

It is obvious that in addition to the quantity of filter film and the residence time in the filter, oxygen transfer, and temperature play a role in filter performance. Oxygen is transferred to the filter film from the waste liquid passing through the filter. The filter slime receives additional oxygen by diffusion through laminar films of liquid remaining in the filter after the applied liquid has drained away. In some cases additional oxygen is supplied through forced draft ventilation. The rate at which oxygen is absorbed by waste passing through a filter has been related to the saturation deficit, the depth of the filter, the hydraulic loading to the filter, and physical variables characteristic of the filter (22).

$$\frac{dC}{dD} = \frac{K'}{Q^{0.5}} \left(C_s - C\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

The quantity of active film will depend on the depth of film through which oxygen will penetrate to maintain aerobic conditions. This film depth can be developed from the relationships to follow.

In the trickling filter during steady state operation the rate of oxygen transfer from the flowing liquid to the film can be approximated by

$$M = \frac{D_L}{h} A \left(C_1 - C_2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (11a)$$

in which C_1 and C_2 are the oxygen concentration at film depths 1 and 2; D_L is the oxygen diffusivity through the film and h represents film thickness.

The rate of oxygen utilization by the film may be expressed

$$M = K_r \rho A h \quad \ldots\ldots\ldots\ldots\ldots\ldots (11b)$$

in which K_r is the oxygen utilization rate of the film, and ρ represents specific gravity.

Under steady state conditions, the rate of oxygen consumption by the film will equal the rate of oxygen transfer to the film and

$$\frac{D_L}{h} A \left(C_1 - C_2\right) = K_r \rho A h \quad \ldots\ldots\ldots\ldots\ldots\ldots (11c)$$

The maximum penetration of oxygen will occur when the oxygen concentration C_2 at a depth h is O and

$$h = \sqrt{\frac{D_L C_1}{K_r \rho}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11d)$$

EFFECT OF TEMPERATURE

Temperature exerts an influence on all biological processes. Many investigators have shown, however, that the temperature influence on trickling filters is considerably less than that predicted by conventional formulas for biological oxidation. This deviation can be rationalized when filter operation is considered as a dynamic equilibrium between several rate equations. It is probable in summer operation that the oxygen transfer mechanism may exert a controlling influence due to increased respiration rates and decreased oxygen solubilities. In winter operation the respiration rate may control. In such a case the temperature dependency will be considerably less than that predicted from previously derived equations. C. J. Schroepfer (23) observed a variation in BOD removal efficiency of 0.18% per °F temperature change. Of significance is the fact that a sewage or water temperature drop of 4°F to 6°F may correspond to as much as a 52°F change in air temperature. W. E. Howland, M. ASCE (24), has shown that the temperature effect on filter performance may be expressed by the relationship:

$$E = E_{20}\, 1.035^{(T-20)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

A possible mechanism to define the temperature effect on trickling filter efficiency can be illustrated by the following example.

Example 3.—At 30°C the oxygen uptake rate of filter film is 20 mg O_2 per hr per ml slime. The average dissolved oxygen is 2.0 ppm; at 20°C the dissolved oxygen is 4.0 ppm; compute the temperature coefficient on the ultimate filter efficiency.

The temperature effect on the oxygen uptake rate of the filter film is

$$K_r = K_{20}\, 1.08^{(T-20)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

The thickness of aerobic film may be approximated

$$h = \sqrt{\frac{D_L C}{K_r}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

At 30°C

$$h = \sqrt{\frac{0.097\ (3.0)}{1000\ (20)}}$$

At 30°C

$$D_L = 0.097 \text{ sq cm per hr}$$

Therefore $h = 3.83 \times 10^{-3}$ cm of aerobic film. At 20°C

$$D_L = 0.074 \text{ sq cm per hr}$$

$$K_r\ (20°C) = \frac{20}{2.15} = 9.3 \text{ ppm per hr per ml}$$

$$h = \sqrt{\frac{0.074\ (4.0)}{1000\ (9.3)}}$$

$$= 5.7 \times 10^{-3} \text{ cm of aerobic film.}$$

Total oxidation at 30°C; (per sq cm)

$$\left(3.83 \times 10^{-3} \text{ cu cm}\right) \frac{20 \text{ mg}}{(\text{hr})\ (\text{cu cm})} = 0.0765 \text{ mg per hr.}$$

At 20°C

$$5.7 \times 10^{-3}\ (9.3) = 0.053 \text{ mg per hr}$$

Temperature Coefficient = 1.038

CONCLUSIONS

The performance of the trickling filtration process has been formulated assuming that BOD removal is related to hydraulic loading and depth and follows a retardant type reaction. Multiple regression analysis on various published filter data has indicated that the effects of hydraulic loading and depth on BOD removal vary with the type of filter, temperature, and so forth. A relationship has been developed using average coefficients for the design of trickling filters treating domestic sewage. A hypothesis has been advanced to describe the effects of oxygen and temperature on filter performance.

It is recognized that considerable fundamental work needs to be done to more closely define filter performance characteristics.

ACKNOWLEDGMENTS

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APPENDIX I.—NOTATION

A = surface area of filter film;

C = constant;

C_1, C_2 = concentration of oxygen in filter film;

C_s = oxygen saturation value;

D = filter depth;

D_L = oxygen diffusivity of filter film;

E = BOD removal efficiency;

h = depth of filter film;

K = modified rate coefficient;

K_r = unit oxygen utilization rate of filter film;

k = BOD removal rate coefficient;

L_a = BOD of settled sewage or waste;

L_e = BOD concentration in filter effluent;

L_o = BOD concentration applied to the filter;

M = mass of oxygen transferred to filter films;

m = exponent on filter depth;

N = recirculation ratio;

n = exponent on hydraulic loading;

Q = hydraulic loading per unit filter surface;

T = temperature, in °C; and

t = time of contact; and

ρ = specific gravity of filter film.

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SANITARY ENGINEERING EDUCATION IN A CHANGING WORLD

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SYNOPSIS

Engineers have a social, as well as a technical responsibility, to help control the environment for the "well-being of mankind," and their education should prepare them for this role. The history and status of technological education in the United States, Continental Europe, and Great Britain are compared. Sanitary engineering education, while further developed in the United States then in Europe or Britain, still suffers from many shortcomings. With its disappearance at the undergraduate level, too few graduates are studying at the master's or doctorate level. Some seventy curricula are available, but fifty of them attract no more than one or two students each year.

Graduate sanitary engineering education is expensive, requiring strong support particularly in the sciences, chemistry and biology, and these sciences for sanitary engineers should be taught by sanitary chemists and biologists in courses designed specifically for sanitary engineering students. It is also a responsibility of educational institutions to recruit qualified chemists and biologists for teaching, research, and operations in the sanitary engineering field. A plea is made for the incorporation of humanistic social science courses in the education of sanitary engineers.

Research in educational institutions improves teaching, recruits and prepares sorely needed research workers, and makes substantial contributions to knowledge. Some of the current research problems in sanitary engineering are discussed. With a chronic shortage of sanitary engineers, opportunities

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abound for young men in this field. Government at all levels, including the international, industry, consultants and educational and research institutions all need increasing numbers of sanitary engineers. And today in the United States, qualified engineering graduates can obtain financial support for graduate study in sanitary engineering. Such support is recognized to be a government responsibility.

INTRODUCTION

To survive in the world, men must accomplish two important tasks: they must control the forces in nature and the forces in man. The first task may be defined as the work of the engineer, and the second as the work of society, not excepting engineers. Sanitary engineers have some responsibility for resolution of social problems, if only to permit work on the environmental problems.

Our natural resources are polluted by radio-active residues of nuclear bomb tests, while engineers are seeking ways to cleanse our air, water, and food. Engineers have an equal, and perhaps even greater, responsibility to control men's despoilation of these resources. In some communities people do not hesitate to foul their own water and air. Sanitary engineers try to protect man by ingenious methods of water and air treatment, and physicians try to ameliorate the effects of these indecencies by adjusting man to his environment. Engineers have the obligation, as participants in society, to try to prevent these insults to the environment in the first place.

How engineers can discharge this obligation to society in the fullest sense, and not in only narrow technical service is important.

A definition of engineering adopted by the Institution of Civil Engineers (I.C.E.) in Great Britain more than a century ago states: ". . .the art of directing the great sources of power in nature for the use and convenience of man." Sanitary engineering requires only a slight modification of this definition: ". . .for the health and well-being of man."

Sanitary engineering as a distinct professional field has had its longest history in the United States, and a definition of the "Sanitary Engineer" proposed by the U. S. National Research Council in 1954 may serve as an introduction to his present responsibilities and field of action. The "Sanitary Engineer" is an engineer ". . .who has fitted himself by specialized training, study and experience (a) to conceive, design, appraise, direct, and manage engineering works and projects developed as a whole, or in part, for the protection and promotion of the public health, particularly as it relates to the improvement of man's environment, and (b) to investigate and correct engineering works and other projects that are capable of injury to the public health by being or becoming faulty in conception, design, direction or management."

The term "public health" as used in the definition has itself taken on new meaning. Under the aegis of the World Health Organization (W. H. O.), public health has come to signify not merely the absence of disease or infirmity, but rather a positive state of complete physical, mental, and social well-being. This philosophy is not new to the engineer who has infused his water supply projects with qualities much broader than the prevention of disease alone. In fact, a definition currently proposed (1961) by the Engineers' Council for Professional Development (ECPD) in the United States for all engineers uses the expression "progressive well-being of mankind."

These definitions serve to highlight the sanitary engineers' responsibility in managing the environment as a whole, and not only the wet part of it. The National Research Council goes on to elaborate on the fields of activity in sanitary engineering in addition to the traditional areas of water supply and sewage disposal. Included are waste reclamation, control of water and soil pollution, the sanitation of food stuffs, the sanitation of houses, institutions and recreational places, vector control, atmospheric pollution control and the control of radiation in the environment. The W.H.O. definition of environmental sanitation also includes virtually all these areas of activity.

This increased scope of sanitary engineering is not the result of the acquisitiveness of "empire builders." Rather, it is the answer to the need for responsibility in controlling the total environment and recognition that the environment cannot be segmented into water, sewage, refuse, air, etc. A water supply is not developed without thought of waste disposal in the region, nor is an outfall located or a sewage treatment plant designed without careful consideration of all the uses to which the receiving body of water is likely to be put. A solution to a refuse disposal problem can create an air pollution problem, and a solution to an air pollution problem can create a water pollution problem. The control of radioactive materials involves every aspect of the environment.

Not only do the various elements of the environment impinge one on the other, the principles and philosophies of environmental control are much the same for all phases of the environment. The similarities in controlling water pollution, air pollution, food contamination, and radiation are much more cogent than their differences. The need to consider the environment and its impact on man as a unity is becoming daily more evident, particularly in the developed countries of the world. Just as the pressure on the shell of a gas cylinder increases as more molecules of gas are compressed into it, so too, the pressure on our environment increases as more people are crowded into the same space. When the gas in the cylinder is heated, the molecules move faster and the pressure increases to a point at which the cylinder may burst. So, too, people have been invested with higher velocities and with more energy than they may know how to use intelligently. The result, great stress on our environment, and reaction stress on the people in it. The strains may be greater than man can survive.

DEVELOPMENT OF TECHNOLOGICAL EDUCATION

The first university professions were law, medicine, theology, and philosophy. Science ffirst found a place in universities as a service to the physician. Leyden was an important center for the study of Newton's physics in the 18th century. The English universities, under the cloak of the Church of England, were aloof to the new sciences, but the Scottish universities early bore the imprint of Dutch teaching.

The industrial revolution did not begin with the help of the universities. In fact, it was the French revolution that was responsible for government support for science and the opening of the Ecole Polytechnique in 1794. The German state universities with their idea of "wissenschaft," the search for knowledge, were fertile ground for research in science as previously in philosophy. By the end of the 19th century they were the beacons for all who would pursue science. But the university professor and his disciples, free to pursue knowledge for its own sake, did not concern themselves with applications of science.

The needs for education in technology were met by the later creation of the Technische Hochschulen, that quickly achieved high status in all the countries of Europe.

Meanwhile, English universities were slow to accept science and technology, and engineers were prepared entirely by apprenticeship in practice. The need for advanced technology to permit Britain to compete with continental countries, the rise of democracy and a managerial class in Britain, and the impact of science itself, all served to inject science and technology into the university structure. This, combined with the British university's devotion to the student as contrasted with the German view of the student as a perhaps necessary impediment to research, resulted in a different structure for scientific and technological education in Britain.

The hard-headed way in which the British went about improving their technological education is revealed in an 1870 report.[2] Several excerpts from this report are of interest. "It is not the custom in England to consider theoretical knowledge as absolutely essential. . . The practical education in England is perhaps the most perfect possible . . . This thorough proficiency in practical matters tends largely to compensate for an in many cases to outweigh the deficiency in theoretical attainments."

On the other hand, a high degree of theoretical knowledge was mandatory for engineers on the continent, after which practical training was acquired. These antithetical approaches were developed in the report,[2] with a conclusion in favor of integrating science in the education of British engineers. That this conclusion was reached may be attributed to the comments of a Henry Conybeare, whose words have validity today. In a letter to the I. C. E., dated August 23, 1868, he wrote:

> "I gave a good deal of attention to the subject (of professional education) about twelve years ago.
>
> At that time my impression of the respective results of Engineering education of the continent and of this country was, that foreign Engineers were usually wanting in practical experience, and our own in scientific training. But I think that this is no longer the case, for the large amount of work that has been executed in France during the last decade has afforded to the French Engineers, opportunities (of which their previous scientific training enabled them to avail themselves to the utmost) of acquiring the practical experience in which they had been before wanting, . . ."

So today, engineering science and technology is a responsibility of the British university. While the apprenticeship system still exists in Britain, more than 80% of engineers admitted to the Institution of Civil Engineers now come through the universities or their equivalent.

In the United States engineering education developed first in the United States Military Academy. The greatest impetus to engineering education in the United States was given by the Morrill Act, which in 1862 made grants of lands to each state for the establishment of colleges to teach agriculture and the mechanical arts, as engineering was then known. In a decade, the number

2 "The Education and Status of Civil Engineers in the United Kingdom and in Foreign Countries," by I.C.E., London, 1870, p. 213.

of engineering institutions grew from six to seventy. These land-grant colleges, particularly in the midwest and west, count among them the biggest and best of United States engineering schools.

Daniel Calhoun,[3] in a study of the early development of Civil Engineering in the United States sums up the role of the engineer in these questions that were being then put by society . . . "Can the project proposed be done?" and "will its utility warrant the cost?" These questions are still pertinent.

The answer to the first question requires scientific and technical competence, but does not constitute the main contribution of engineering. The assessment of the utility of a project and its cost, for society to weigh and make the intelligent choice, constitute the unique charge to the engineer. And it is clear that the exercise his responsibility he needs to recognize the impact of his project on society. Therefore, the role of engineering education is broader than science and technology and this is especially significant in the education of sanitary engineers, whose projects are woven into the fabric of every level of society's development, from the most recently emerged nation, to the most highly developed industrial community.

STATUS OF SANITARY ENGINEERING EDUCATION

Sources of information on engineering education are the reports of the European and United States Engineering Conference (EUSEC), that are being prepared at the headquarters of The Netherlands Royal Institute of Engineers.[4,5] A W.H.O. volume[6] and a Dutch study[7] are particularly useful with regard to sanitary engineering.

In the United States, secondary school education is generally at an extremely low level when compared to secondary schools in Europe, particularly with regard to mathematics and science. Much of the first years of the conventional 4-yr engineering program leading to a Bachelor of Science in Engineering degree in the United States is spent on subjects that are covered in European secondary schools. It has been said that students from good secondary schools in Europe may be admitted directly to the second or third year of some American engineering institutions.[8] It is clear that the graduate of the American 4-yr engineering school has not achieved the same level of technical education as the continental graduate who, beyond strong secondary school preparation, takes at least 5 yr, the last year being given to an area of speciality and a project that the student develops independently.

The British engineering course is 3 yr, but as the preparation for entrance involves much of the science that may be taken in the first year at an American institution, the graduate is at about the same level as his American counterpart.

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3 "The American Civil Engineer, Origins and Conflict," by Daniel Hoven Calhoun, Tech. Press, MIT, Cambridge, Mass., 1960.

4 "Education and Training of Professional Engineers," EUSEC Report in preparation at the Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

5 Proceedings, 3rd EUSEC Conf. on Engrg. Education, Paris, September, 1957, I.M.E., London, 1958.

6 "The Training of Sanitary Engineers," by Milivoj Petrik, WHO, Geneva, 1956.

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While the graduate of the American and British engineering school is at a somewhat lower level than the continental, if he completes a graduate program, that generally takes 1 yr, he may be more advanced. The graduate program generally provides a more intensive and directed education than the fifth year of the continental undergraduate program. However, only since the war has there developed in the United States and the United Kingdom the concept that graduate education is important in the education of engineers, particularly those who aspire to professional attainments.

In Britain, in 1958, for example, only 150 students were enrolled in post-graduate programs in civil engineering, 1/3 of whom were British. It was estimated that fewer than 1 civil engineering graduate in 15 goes on to graduate study.[9] In the United States about 20% of civil engineering graduates earn Master's degrees. This is a 100% increase over the pre-war percentage, but is still far from adequate when it is recognized that undergraduate education for engineers in the United States, emphasizes science and engineering science, with little attempt at specialization or engineering practice. University graduates in engineering, particularly those who exhibit qualities that mark them for leadership in the profession, are today being urged to go directly into graduate study at least to the Master's degree. Funds are becoming increasingly available for supporting graduate students so that it is quite fair to say that in the United States any qualified graduate of an engineering school can find the financial support he needs to undertake graduate study.

In sanitary engineering, the trend to graduate study has been particularly strong. This trend was formalized and given impetus by the Conference on Education, Training and Utilization of Sanitary Engineers in 1957, which recommended a 5-yr program, involving 4-yr undergraduate engineering and professional specialization in, the fifth year leading to the Master's degree.[10] Since World War II, undergraduate programs of sanitary engineering, even as options in civil engineering with little specialization, have been abandoned. From 1950 to 1959, for example, the number of men receiving the Bachelor of Science degree with major in sanitary engineering, dropped from 287 to 182. In the same period, the number of Master's degrees awarded increased from 148 to 197. On the continent, where graduate education is not general in engineering, only a few institutions offer formal graduate curricula in sanitary is insignificant. The W.H.O. symposium on the education of sanitary engineers, held at Oxford in 1955, emphasized the need for specialized education.[11] The new European Course in Sanitary Engineering at the Technological University in Delft is undoubtedly a response to that need.

Financial support for graduate study in sanitary engineering has had a salutary effect on recruitment. Government service, that utilizes many sanitary engineers, particularly at the local level, is not able generally to compete with industry and construction. The availability of stipends for graduate study in sanitary engineering, however, is helpful in winning students to sanitary engineering, from which many are likely to go into public service.

With the increase in graduate study in engineering, and a concomitant increase in research and the need for engineering educators, the doctorate has

9 "The Place of the University in the Education of Civil Engineers," Proceedings, I.C.E. London, p. N5, April, 1959.

10 "Education, Training and Utilization of Sanitary Engineers," Conf. Report, Natl. Academy of Sciences, Natl. Research Council, Washington, D.C., 1957.

11 "Symposium on the Education and Training of Sanitary Engineers in Europe," WHO, Geneva, 1955.

attained a more significant place in engineering education. Whereas the Master's degree is generally terminal for professional preparation, those who aspire to academic life, particularly graduate teaching and research, today must attain a doctorate. Currently, about 10% of those who earn Master's degrees continue on to earn the doctorate, with the percentage being somewhat higher in sanitary engineering than in other specialities of civil engineering. Where, before World War II, 1 or 2 men earned a doctorate in sanitary engineering each year in the United States, this has now increased tenfold. The doctorate for sanitary engineers is rare in Britain, and non-existent on the continent except perhaps in Germany.

A word might be interpolated on the relative earnings of men with these various degrees. In the United States today, (1961) men with a Master's degree without experience will earn about 10% to 20% more than the man with the Bachelor of Science, and the doctorate will bring in another 60% to 70%.[4,5]

Among sanitary engineers in the United States today, it is estimated that about half hold the Mater's degree. It is hoped that soon, in the United States, almost all sanitary engineers will enter the profession with the Master's degree. This may be formalized by making a Master's degree a prerequisite for certification as a Diplomate of the American Academy of Sanitary Engineers.

It might well be asked, if all sanitary engineers are to take graduate work or a fifth-year, and preferably directly after graduation with the B.S. degree, that this program be added to the undergraduate program at each institution in order to graduate the man with his year of specialization. (On the continent this is already done, except that sanitary engineering is not often one of the specialities available.)

Sanitary engineering requires a great breadth of education. It is expensive and beyond the resources of many engineering institutions. In this United States many small engineering schools graduate 10 to 20 in civil engineering each year. Most of these have no graduate or specialized programs at all, and the inauguration of sanitary engineering would be far too expensive. Whereas some 140 institutions in the United States have accredited undergraduate programs in civil engineering, about half this number have graduate curricula in sanitary engineering. Of these seventy, in 1959, fifty institutions awarded fewer than 3 Master's degrees. At present (1961) only undergraduate curricula are accredited, but consideration is being given by the American Sanitary Engineering Intersociety Board to accreditation of graduate sanitary engineering programs.[12] In Britain, of 7 institutions offering graduate work in civil engineering only 3 offer curricula in sanitary engineering.[9]

CONTENT OF SANITARY ENGINEERING EDUCATION

It is not the purpose here to discuss curriculum in terms of hours or even percentages of time. The inevitable curriculum committee will have to wrestle with these problems. The total time available is fixed, and the pressure to incorporate new knowledge that is "indispensable" is very great. However, hours of subject matter are often not indicative of the values to be obtained. One hour at the foot of an inspired teacher, no matter what the subject, may be more valuable to the student than 20 hr of pedestrian teaching in his major

12 Report of Amer. San. Engrg. Intersoc. Board Study Conf. on Graduate Education of San. Engrs., Cambridge, Mass., June 27 to 29, 1960.

field of interest. Programs should be flexible so that they may make use of rare talents that are available at most institutions.

The Sciences and Mathematics.—Glenn T. Seaborg, Nobel prize-winning discoverer of plutonium, has stated "Science should be a part of the repertory of the cultured man." How much more should this be true for the engineer, who applies science. Science provides the engineer with the foundation on which he can build future research and developments into engineering practice. The discipline instilled while studying a quantitatively described branch of knowledge helps him to weight evidence, analyze problems, and synthesize solutions throughout his engineering career. It may also stimulate him to be among the small number who will advance the profession by engaging in research. What distinguishes science for sanitary engineering is that all the natural sciences are involved.

An engineer should acquire his mathematics through differential equations early in his education in order to permit physics and chemistry to be learned in terms of the mathematics on which they are built. In sanitary engineering fields, the gap between cause and effect is often great and correlation can only be established by applying sound statistical principles, so that study in statistics is essential.

All engineers must have course work in physics, including electricity and nuclear physics. The engineering sciences common to all engineering study include solid and fluid mechanics, the nature and properties of matter including strength of materials, thermodynamics, and transfer and rate mechanisms. The applications of these sciences in sanitary engineering are too obvious to need elaboration. In addition the civil and sanitary engineer needs preparation in geology and hydrology.

In order to understand the principles that apply in almost every area of sanitary engineering, the engineer should have preparation in inorganic chemistry, organic chemistry, and physical and colloidal chemistry. New knowledge in the chemistry of chlorine solutions has modified our disinfection practices. The continual evolution of new synthetic chemicals has presented new tools as well as new headaches. As much of his work involves very dilute concentrations of substances in water, food, and air, the sanitary engineer needs special study in analytical methods including instrumental analysis and radiochemistry. While sanitary engineers do not need the breadth or depth in chemistry that chemists require, they should have a working comprehension of the fundamentals applicable to sanitary engineering problems.

Control of the environment for the protection of health involves a fundamental understanding of the physiology of living organisms - from the smallest to the human. Bacteriology and virology, parasitology, physiology, epidemiology, entomology, hydrobiology, and ecology are all necessary studies for the sanitary engineer. An understanding of the stabilization of wastes, whether in natural or in artificial environment, involves an appreciation of biochemistry. Again, the breadth and depth in these sciences need not be what biologists require, but special courses can be designed to include the areas partinent to sanitary engineering.

Undergraduate engineering curricula are still far short of what is necessary to provide the basic natural sciences required for specialized graduate study. Therefore, some essentially elementary science material must be given in the graduate program. However, this can be presented at a rapid rate and tailored to meet the particular needs of the sanitary engineer. In time, the basic sciences may all be covered in the earlier years.

L. W. Grinter[13] points out that as a result of the emphasis on science in the 1955 Report on the Evaluation of Engineering Education,[14] about half of all United States engineering institutions have increased their physics, mathematics, and engineering science offerings, but only about 10% have increased chemistry. No mention at all is made of biology, a subject that is included in few curricula for engineers.

The Engineering Courses.—Sanitary engineering developed as a field of specialization in civil engineering. Today with increasing industrialization, more chemical engineers and some mechanical engineers are being drawn into the field. In fact, both the American Society of Civil Engineers and the American Institute of Chemical Engineers are represented on the American Sanitary Engineering Intersociety Board (ASEIB). Still, by far the greatest number of sanitary engineers in the United States continue to be drawn from among civil engineering graduates.

All civil engineers will generally need courses in structural design, hydraulics, foundation or soils engineering, and water supply and drainage. However, it is not the courses themselves that are so important but rather the concept of engineering analysis and design. The principles involved in the planning, analysis, design, and execution of a project in one field are certainly useful in another. And this is to be preferred to survey and standards of practice courses in which the emphasis is on current technique. Even when engineering methods are available, there is room and need for creativity, vision, and imagination. The time to master standardized procedures, the rule of thumb, is in practice where the sheer volume of day-to-day work does not permit continuous modification of method. Here, too, the mark of the professional is not repetition, but creation. Each job should provide lessons for the next and it is this attitude that must be cultivated in the classroom.

On the other hand, there is a tendency to frown on practical work, and to devote the students' time to science and survey. The engineer can learn much from getting his feet wet and his hands dirty. An engineer must make decisions. He should begin to learn the cost of decision-making early in his professional education.

Sanitary Engineering Courses.—The role of the sanitary engineer is today so broad, that it is impossible for any single person to be well qualified in all phases of environmental control, much less to encompass the technical material in a one-year program. The American Academy of Sanitary Engineers allows for specialization in one of several areas in applying for certification: water and waste disposal, air and industrial hygiene, environmental sanitation, and radiological health. The intention in most graduate programs is to explore one area in some depth, and touch the others lightly. Only a few institutions in the United States are prepared to teach in all areas, and certainly none is equipped to teach in depth in all areas. This paper explores the most common field of interest to the sanitary engineer, namely, water supply and waste water disposal. Perhaps the simplest way to illustrate the program is to use the sanitary engineering Master of Science in Sanitary Engineering program of a Department of Sanitary Engineering at a School of Public Health in the United States. The courses shown each involve 30 to 45 lecture hours, and

13 "A Survey of Current Changes that are Modernizing Engineering Education," by L. W. Grinter, Journal of Engineering Education, Vol. 44, p. 559, March, 1959.

14 "Evaluation of Engineering Education," Report, Journal of Engineering Education, Vol. 46, p. 25, September, 1955.

where laboratory and design are indicated, these involve an additional 30 hr to 45 hr. Generally about 12 courses are taken and a report on a special project is executed. Students must pass all courses taken as well as a written comprehensive examination.

Required: Epidemiology
Parasitism and human disease (with lab)
Sanitary chemistry and biology
Analytical methods in sanitary chemistry and biology (with lab)
Chemistry and biology of water and waste treatment processes (with lab)
Water supply and sewerage (with engineering analysis and design)
Water purification, sewage treatment and disposal (with engineering analysis and design)

Electives generally taken:
Radioactivity in water, food and air (with lab)
Limology and stream pollution (with lab)
Industrial water supply and liquid wastes

Other electives:
Principles of statistical inference
Environmental sanitation methods (with lab)
Sanitary microbiology (with lab)
Fundamentals of air hygiene (with lab)
Industrial hygiene and sanitation (with lab)
Public health administration

Students may also take work in other departments of the university, such as in City and Regional Planning.

A Master's degree curriculum in a Department of Sanitary Engineering or a Department of Civil Engineering in an engineering school would be little different from that previously described except that the electives, for which there is little time in a 1 yr program, might be selected from engineering courses.

The only thing that can be said with certainty about this program is that it is continually changing. The important consideration is not a fixed program for all institutions, but rather freedom for each instituion to draw on its unique strengths in creating a suitable program. A university is not a collection of courses, facts, libraries, or laboratories - it is a collection of individuals each dedicated to a specific field of learning and each confident that his speciality is of prime significance in the curriculum.

Many subjects have been listed and with the high degree of specialization today, a great number of different teachers can be used. However, this is neither reasonable nor desirable. Whereas a student majoring in biology would expect to have a different specialist teach each of the courses listed under the biological sciences, the biology for the sanitary engineer might be handled by one man - but he should be a biologist with a deep interest in one of the areas of biology significant to sanitary engineering. Biology is one of the sciences inadequately covered in most engineering curricula. One of the ways in which this deficiency could be met, if time were available in the curriculum, is by sending the students over to the biology department. However, these courses are generally designed for students who will then take other, more advanced courses. For the engineer this course might be his first and

last contact with the subject. This is why courses should be designed specifically for the sanitary engineer, particularly when given at the graduate level. To have all these specialists available to a sanitary engineering program makes sanitary engineering education expensive.

Humanities and Social Sciences.—The two most common methods for including the humanities in education of engineers in the United States is through the increasingly popular 2 yr to 3 yr sequence, in which the student takes 2 yr of humanities in a so-called liberal arts college, and then 3 yr in the engineering school. Edwin S. Burdell[15] calls this the "pre-heat" treatment. A more common method is the inclusion of one non-engineering course in each semester in engineering school. Burdell characterizes this as the "sheep-dip" treatment. On the continent, humanities courses are often part of a "general study" sequence, given generally at the end of the day, and entirely optional. This might be termed an application of a "veneer" that at the end of the year, gets to be thin.

This presentation began with emphasis on the need for the engineer to be aware of the social consequences of his work. Eric Ashby[16] puts it very well:

> "Technology could become the cement between science and humanism. Far from being an unassimilated activity in universities, it could become the agent for assimilating the traditional function of the university into the new age. For technology is inseparable from men and communities. In this respect technology differs from pure science. It is the essence of the scientific method that the human element must be eliminated. Science does not dispense with values but it does eliminate the variability of human response to values. It concerns itself only with phenomena upon which all qualified observers agree. It describes, measures, and classifies in such a way that variation due to human judgement is eliminated. Unlike science, technology concerns the applications of science to the needs of man and society. Therefore technology is inseparable from humanism. The technologist is up to his neck in human problems whether he likes it or not. Take a simple example: the civil engineer who builds a road into a new territory in tropical Africa. He may assert that it is not his business to take into account the effect his road will have on primitive villages up-country; but his road is in fact a major experiment in social anthropology. He does not need to be a professional anthropologist, but he cannot afford to be utterly ignorant of the implications of his work."

W. E. Wickenden,[17] of the Committee on Engineering Education after World War II, was outspoken in his advocacy of the humanistic-social and scientific-technological curriculum. The committee recommended that humanistic-social courses constitute at least one-fifth of the total curriculum. However, he pointed out that

> "We do not justify our position if we make our humanistic-social program a collection of oddments and remainders, a bit of this and a smattering of that, demanding casual effort and superficial achievement. We

15 "General Education in Engineering," by Edwin S. Burdell, Journal of Engineering Education, Vol. 47, p. 103, October, 1956.

16 "Technology and the Academics," by Sir Eric Ashby, Macmillan and Co. Ltd., London, 1959.

17 "Engineering Education After the War," Report, Journal of Engineering Education, Vol. 35, p. 36, September, 1944.

do not justify our position if we relegate these responsibilities to routine teachers . . . We do not justify our position if we let teachers of humanities regard us as Philistines and excuse their efforts on our behalf as a missionary enterprise."

"To be blunt about it," he adds, "it is now time to put up or shut up - to give the humanities and the social studies the time, the leadership, the manpower, the content and the financial support that will put them on a level in prestige and excellence with our scientific disciplines."

In a study on technological education, UNESCO[18] concluded that "Universities . . . cannot be satisfied with offering only professional training. To turn out technicians is only a part of their task. A living community also needs leadership and in a democratically organized educational system, it is from the ranks of the Universities that this leadership must come."

Progress has been made, and there are a few outstanding examples of integration of the humanities in the engineering curriculum.[19] Of these, it can confidently be said that the engineers' education is equal to any. But in most instances a block of time is given, the engineering student is sent over for his injection of culture, and that's it. At the graduate level, the attempt at introducing humanistic social courses is seldom made. The integration of humanistic courses in a technological curriculum requires imagination and effort if the student is to profit and not feel that it is a routine obligation, like vaccination or a foreign language. Again Eric Ashby puts it well:

"The habit of apprehending a technology in its completeness: this is the essence of technological humanism, and this is what we should expect education in higher technology to achieve. I believe it could be achieved by making specialist studies (whatever they are: metallurgy or dentistry or Norse philology) the core around which are grouped liberal studies which are relevant to these specialist studies. But they must be relevant; the path to culture should be through a man's specialism, not by by-passing it. Suppose a student decides to take up the study of brewing: his way to acquire general culture is not by diluting his brewing courses with popular lectures on architecture, social history, and ethics, but by making brewing the core of his studies. The sine qua non for a man who desires to be cultured is a deep and enduring enthusiasm to do one thing excellently. So there must first of all be an assurance that the student genuinely wants to make beer. From this it is a natural step to the study of biology, microbiology, and chemistry: all subjects which can be studied not as techniques to be practised but as ideas to be understood. As his studies gain momentum the student could, by skilful teaching, be made interested in the economics of marketing beer, in public-houses, in their design, in architecture; or in the history of beer-drinking from the time of the early Egyptian inscriptions, and so in social history; or, in the unhappy moral effects of drinking too much beer, and so in religion and ethics. A student who can weave his technology into the fabric of society can claim to have a liberal education; a student who cannot weave his technology into the fabric of society cannot claim even to be a good technologist."

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18 "Education in a Technological Society," UNESCO, Paris, 1952.

19 "General Education in Engineering," Report, Journal of Engineering Education, Vol. 46, p. 623, April, 1956; Vol. 51, p. 141, November, 1960.

This example is broadly drawn, but if applied to sanitary engineering it might help to avoid blunders, such as the well-designed water and sewage projects that are overloaded a year after they are placed in operation. A good system of water supply and sewerage stimulates the growth of a community and it is no longer the same community after the works are completed.

The great problem in metropolitan areas today is not where to obtain water or how to treat it, or how to collect waste, how to treat it and where to dispose of it. The technical competence is available. But how shall the communities in the metropolitan area be organized to finance the projects? How can the communities be induced to forego local sovereignties and join together to solve their engineering problems, so that one community's solution does not become another comminity's headache? This problem exists in Europe on an international scale with the River Rhine. It is clear that the cleansing of the Rhine is much more than a technical sanitary engineering problem.

To sum up, sanitary engineers need to be truly educated and for this purpose their humanistic-social courses must be integrated with their technical and scientific courses.

RESEARCH IN EDUCATIONAL INSTITUTIONS

The principal function of the early universities was the accummulation of new knowledge. As undergraduate education expanded, research tended to be forgotten, and today in many educational institutions the faculty engages in no research.

Hardy Cross[20] points out that "It is not fair to feed students entirely on secondhand information. College instructors should be able sometimes to tell their students that such and such is true because they saw it in their own laboratory!" In the applied areas, first-hand information is often available, particularly in the Technische Hochshulen where the faculty is drawn from among leading engineering practitioners. Engineering institutions, however, also have a responsibility for the accumulation of new knowledge on which practices are based.

The value of research in educational institutions was discussed in a paper presented at the Seventh European Seminar for Sanitary Engineers in Madrid,[21] and a few of the comments are pertinent here:

> "The contribution of research to teaching lies in the quality of the teaching. The professor who is acquiring new knowledge is more likely to be knowledgeable of advances in allied fields. He will pass along to his students the awareness that answers are just being developed and that practice is at best only the current solution to engineering problems. Perhaps the most important contribution that research can make to the teaching program is in the creation of a stimulating atmosphere. Particularly today, with rapid development in science and technology, our educational processes must prepare students not for problems of yesterday and the solutions of today, but for the problems and solutions of tomorrow. To obtain our proper quota of bright young students and

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20 "Engineers and Ivory Towers," by Hardy Cross, McGraw-Hill Book Co., Inc., New York, 1952.

21 "Sanitary Engineering Research in Educational Institutions," by Daniel A. Okun, 7th European Seminar for San. Engrs., Madrid, October 6 through 13, 1960.

faculty members in our educational institutions, and more particularly, to compete with the excitement of the nuclear and space age offered to budding scientists and electrical engineers, we in sanitary engineering must not only pose the challenges of our modern environment but we must begin to work on solutions to these problems.

Still another contribution of research is in the education and training of research workers. Every professional group, if it is to maintain its role in society, must add to knowledge. This does not happen by chance. Trained workers oriented to seeking new information must be available. While such workers may often be developed on the job, the introduction of concepts of research early in a man's education might encourage him to pursue a career in research. The educational institution is probably the best place for recognizing research competence on the part of the young engineer and scientist. Government and private laboratories can only select from those who apply and have little opportunity to recognize and stimulate raw talent. These laboratories are primarily concerned and properly so, with the research itself. They cannot afford to invest time, money, and facilities in preparing the young research worker for his career. The recruitment and training of research workers is one of the educator's more important responsibilities today.

The educational institution offers an opportunity to conduct research of a type that often is not or cannot be carried out in government or private laboratories which are generally dedicated to applying our currently available knowledge to the solution of specific problems. The gathering of new knowledge, so-called basic research, is generally left to occasions when the need for solving current problems is not acute. In only rare instances is a government or private research organization involved in basic research. This does not signify that the educational institution engage only in basic research. In many instances, particularly where engineering experiment stations are associated with the institution, many applied problems will be taken on. The instituion's primary responsibility, however, lies in the accumulation of new knowledge.

Another important aspect of the research in an educational institution is that problems selected need not be politically oriented. Many government and private laboratories are forced to handle problems which are currently pressing and which influential people in government believe need to be solved. Private organizations will work on problems which can yield profit or perhaps save money. The problems of the future which are not yet pressing but for which we need information are the concern of the university where the accumulation of knowledge is itself sufficient stimulus and where there is no responsibility to either a government bureau or a client. This points up an important characteristic of the university: it permits its faculty to follow paths which interest them rather than directions dictated by others. This is not a luxury, and universities have been stalwart in maintaining this freedom of inquiry for their faculties."

Graduate study leading to the Master's degree is generally oriented to preparing for professional practice, and in a 1-yr program there is seldom time for research. On the other hand, those interested in research, whether because they intend research careers or eventually to go into teaching, generally take

2 yr for the degree, giving half-time to a research project. And the increasing number of doctoral candidates contribute to the university's research effort. The amount of research in universities in the United States is illustrated by the 1958 Research Inventory of the United States. Public Health Service.[22] Of 280 research projects in water supply and pollution control, 73% were conducted in educational institutions, 8% by federal agencies, 13% by other governmental agencies and 6% by industry. Of 79 research papers in the Journal of the American Water Works Association and the Sewage and Industrial Wastes Journal in 1959, 40 originated in educational institutions. Of 18 Harrison Prescott Eddy Research Medals awarded by the Research Medals awarded by the Water Pollution Control Federation in the period 1943 to 1960, eleven were won by researchers in educational institutions.

The contributions of chemists, biologists, and physicists are even more important to research in sanitary engineering than to teaching. A brief recital of some of the current research problems in sanitary engineering indicates the need for a team of engineers and scientists.

Reclamation of Waste Water.—In areas of dense population and limited water resources, reuse of waste water is already a fact. The chemical problems, particularly the build-up of detergents and dissolved solids, and the physical and hydrological problems, need considerable study. Concomitant is the need for effective and economical methods of biological waste treatment.

Detection and Removal of Viruses and Micro Chemicals from Waters.—In highly developed industrial societies, diseases caused by the contamination of water with enteric bacteria have been replaced as a problem by viruses and organic chemicals, whose effects may not be so dramatic as the older waterborne diseases but may be more widespread. Little is known about the toxic effects of the long-term ingestion of micro-chemicals.

Radioactivity in the Environment, Its Measurement, Removal, and Effects.—The level of radioactivity in the environment may not yet be cause for concern, although this is debatable. However, nuclear energy and radioisotopes are being used at an increasing rate. The situation with regard to industrial and municipal wastes, in which the damage to the environment has already occurred, should be avoided. The deleterious effects of radioactivity on humans are cumulative, and damage caused by exposure to a contaminated environment cannot later be undone by remedial measures.

An exhaustive list of research needs in all areas of sanitary engineering can be found in the Proceedings of a Conference on "Man versus Environment" held in Washington,[23] in 1958. The need for more research personnel, sanitary engineers as well as chemists, biologists, oceanographers, hydrologists and others was emphasized throughout the conference. This was reiterated at the National Conference on Water Pollution,[24] in 1960. The training and education of sanitary chemists and biologists in the numbers required are the responsi-

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22 Water Supply and Water Pollution Control Research Inventory, Active Projects, 1958, U.S. Dept. of Health Education and Welfare, Pub. Health Service, Washington, D.C., 1960.

23 "Man versus Environment," Proceedings, United States Health Service Conf., Washington, D.C., January, 1959.

24 Proceedings, Natl. Conf. on Water Pollution, December 12 to 14, 1960, U.S. Dept. Health Education and Welfare, Pub. Health Service, Washington, D. C.

bility of sanitary engineering and the place in which this can begin is in the university.

OPPORTUNITIES FOR SANITARY ENGINEERS

A chronic world-wide shortage of scientists and engineers has hampered efforts to control the environment. This chronic shortage will soon become acute as emerging nations in Africa, Asia, and Latin America industrialize. Sanitary engineers have a unique opportunity to give leadership in solving the problems of changing societies and easing the pressures of a world growing more crowded. The specific positions they will occupy are varied, and some have not yet been created.

Sanitary engineers will work, as they always have, in national and local government agencies for the supply of water and disposal of wastes. Government agencies with advisory and regulatory powers, such as health departments that assure the safety of water supply and pollution control agencies, often within the health agency, and that control the quality of public waters, are among the greater users of sanitary engineers.

Industry has found that sanitary engineers are valuable in helping to locate sites for new plants and in planning industrial water supply and waste disposal projects. Sanitary engineers are also used for the control of products in the food industry and for the sale of products in pipe and equipment manufacturing. Sanitary engineers, particularly industrial hygienists, have been widely used for the maintenance of the in-plant environment.

As even small communities are forced to turn to surface waters for water supply, and as all communities will have to provide waste treatment facilities, the need for consulting services will increase. Even large cities often cannot support a permanent staff capable of making detailed investigations and executing detailed designs, so that consultants are required. Qualified consultants have the experience of projects in many cities to draw from, where the staff of a municipality may have had little previous experience with a water or waste treatment project.

Then of course there is the rapidly increasing need for research workers and teachers. Nor have been mentioned other areas of environmental control where sanitary engineers find employment, namely, air hygiene, radiological health, food, and general environmental sanitation.

Among the great opportunities for sanitary engineers is in international health work. Here the engineer, teamed with physician, nurse, social worker, educator, and other health specialists, evaluates health problems in a community or country and helps develop local personnel to meet these problems.

CONCLUSIONS

Growing need exists for the education of sanitary engineers, and for research in environmental health. Both are expensive in terms of the human resources required.

All education must be society's responsibility, and this has been recognized in the provision of publicly supported education at all levels. The educated are a resource and increase the income of a community. The uneducated and untrained often become a drain. However, higher education is still expensive

for the student who aspires to a profession. He will often choose his profession by the financial return and social status he can expect. And these are generally highest in the private or self-employment professions.

Yet much of sanitary engineering cannot be done privately or independently. A citizen may employ a physician to cure him of an infirmity, or perhaps to inoculate him against a disease, but the citizen is helpless in our modern society to protect himself against his environment. He must call on his fellows for joint action. That is the purpose of government and government employs the sanitary engineer.

Similarly in research. Research is generally expected to result in profit - but research in controlling the environment is an investment that can seldom yield a financial return. Therefore, research in sanitary engineering is government's responsibility.

It is up to engineers, as citizens, and as workers in the field of environmental health, to see that our governments, at every level, local, provincial, state, national, and international support sanitary engineering education and research. Individuals and communities are expending large sums and effort that seem to be directed at making our planet uninhabitable. Comparable community effort directed to improving our environment is certainly warranted.

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A SURVEY OF SANITARY LANDFILL PRACTICES

Thirtieth Progress Report
Committee on Sanitary Engineering Research

SYNOPSIS

In the United States, sanitary landfills are a common low cost refuse disposal method; they can provide satisfactory, nuisance-free operation. The general popularity of sanitary landfills is only paralleled by the limited available technical information describing proven operating and land reclamation experience. The technical data presented in this report was obtained from a questionnaire survey of some 250 operating agencies and may be used as a general guide to current sanitary landfill practices.

INTRODUCTION

The sanitary landfill is a popular method of disposing of municipal refuse. Recently there has developed an interest in improving the efficiency of sanitary landfill operations in order to better meet today's and future municipal refuse disposal requirements. The Solid Wastes Engineering Section conducted this study to serve as a guide for better planning and operation of landfill facilities. From 1959 through 1961 over 250 governmental agencies were contacted, primarily by postal questionnaire, and they have cooperated in providing detailed operating experiences and cost information concerning their landfill. Many agencies operate more than one landfill; hence, the reviewed data actually covers far more than 250 sites and describes the procedures representative of the best type of municipal landfill.. The resulting survey data has

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SA 4, July, 1961.

been organized according to population groups served, ranging from less than 10,000 persons to greater than 100,000 persons and is described in Table 1.

COMMENT

Refuse Accepted.—Cellulose is the major component of American refuse; newsprint, magazines, cardboard packaging are examples of processed cellulose materials. Grass cuttings, trees, shrubbery, and garden trimmings are a second source of cellulose type refuse. There are other refuse components

TABLE 1.—QUESTIONNAIRE REPLIES

No. (1)	City or Other Local Governmental Agency Population Groups (2)	Number of Completed Questionnaires (3)
1.	0-10,000	48
2.	10-25,000	63
3.	25-50,000	55
4.	50-100,000	35
5.	Greater than 100,000	25
6.	Unclassified[a]	25
	Total	251

[a] Questionnaires received after completion of tabulation.

TABLE 2.—PROHIBITED TYPES OF REFUSE

No. (1)	Type of Refuse (2)	% of Landfills Prohibiting (3)
1.	Septic tank sludge	81
2.	Flammable liquids	73
3.	Large dead animals	70
4.	Tree stumps	42
5.	Demolition	36
6.	Ashes	11
7.	Rubbish	7
8.	Garbage	7

that are difficult to handle and, in many cases, these are not allowed to be disposed of at landfills. For example: Demolition refuse is generated as a result of the destruction of existing buildings and public works and is a major refuse disposal item; yet, some 11% of the reporting landfills prohibit this material. Over 80% of the reporting landfills prohibit the disposal of septic tank sludge. Other undesirable types of refuse often kept out of restricted fills are shown in Table 2 and include flammable liquids, large dead animals, tree stumps, ashes and garbage.

Refuse Delivery Methods.—Table 3 indicates that over 45% of the landfills accept refuse from both municipal forces and private haulers; whereas, 33% accept refuse from municipal forces only. The other reporting landfills, in

some cases, are operated for the use of only private or contract haulers or combinations of haulers as shown.

Operating Equipment.—The most popular landfill equipment is the crawler tractor with either front end loader or a dozer. This equipment can be used efficiently at low cost for the multipurpose jobs required at a fill including excavation of earth, movement of refuse, compaction of refuse, and for other commonplace construction purposes not necessarily connected with refuse disposal. Additional reasons for the crawler tractor popularity include (a) the universal availability of original equipment and maintenance parts and (b) the relatively high trade-in or resale value. Table 4 also describes the relative popularity of other types of equipment; less than 10% of the reporting fills use

TABLE 3.—DELIVERY BY PRIVATE PARTIES, CONTRACT HAULERS AND MUNICIPAL FORCES

No. (1)	Refuse Delivered By (2)	% of Landfills (3)
1.	Municipal Forces and Private Haulers	46
2.	Municipal Forces Only	33
3.	Municipal Forces, Contract Haulers and Private Haulers	7
4.	Private Haulers Only	6
5.	Contract Haulers Only	4
6.	Contract Haulers and Private Haulers	3
7.	Municipal Forces and Contract Haulers	1

TABLE 4.—EQUIPMENT TYPES

No. (1)	Type of Equipment[a] (2)	% of Landfills Using (3)
1.	Crawler tractor/front end loader	59
2.	Crawler tractor/dozer	50
3.	Draglines	9
4.	Scrapers or Pans	8
5.	Shovels	4
6.	Rubber tired tractor/front end loader	3

[a] One fill each reported the use of: Rubber-tired tractor/dozer; Ripper; Sheeps foot roller; Road roller; Backhoe; and Grader.

such equipment as draglines, scrapers, shovels, or rubber-tired tractors. There is no apparent need for garages to protect landfill equipment except in cold climates; however, continuous preventive maintenance has been determined to be an important measure in reducing operating equipment costs.

The greater the size of the operating agency, the smaller the percentage of operating equipment used for other purposes (Table 5(A)) and also the smaller the percentage of second hand equipment employed at the landfill. As shown in Table 5(B), landfills serving over 100,000 population reported having less than 8% of some items of second hand equipment.

Conversely, as shown in Table 5(C), the larger the site, the less garage facilities provided for equipment. Over 55% of the sites of under 50,000 pro-

vided garages for their equipment; however, only some 20% of the sites of over 100,000 have found a need to maintain garages.

Sanitary Facilities.—Most all job facilities including construction projects are required to maintain sanitary conveniences for employees; it would appear

TABLE 5

No. (1)	Population Group Served (2)	% of Landfills (3)
	(A) Use of Equipment for Other Purposes	
1.	0-10,000	46
2.	10-25,000	38
3.	25-50,000	16
4.	50-100,000	26
5.	Over 100,000	4
	(B) Second Hand Equipment Used	
1.	0-10,000	17
2.	10-25,000	13
3.	25-50,000	9
4.	50-100,000	9
5.	Over 100,000	8
	(C) Garages Provided for Equipment	
1.	0-10,000	77
2.	10-25,000	64
3.	25-50,000	55
4.	50-100,000	49
5.	Over 100,000	20
	(D) Sites Having Sanitary Facilities for Employees	
1.	0-10,000	25
2.	10-25,000	27
3.	25-50,000	35
4.	50-100,000	51
5.	Over 100,000	36
	(E) Water Available Under Pressure At Site	
1.	0-10,000	21
2.	10-25,000	38
3.	25-50,000	38
4.	50-100,000	51
5.	Over 100,000	52
	(F) Telephones Installed at Site	
1.	0-10,000	13
2.	10-25,000	17
3.	25-50,000	31
4.	50-100,000	43
5.	Over 100,000	52

that rest rooms, showers, and locker space are mandatory for "sanitary" landfill operations.

Unfortunately, many of the sanitary landfills do not have available adequate sanitary facilities. The sites having basic conveniences for employees ranged from 25% for sites of less than 10,000 population to 51% for sites of 50,000 to

100,000 population. As shown in Table 5(D), the large landfills are also seriously deficient in maintaining sanitary conveniences.

Water has been demonstrated to be an effective means of controlling flying papers, dust and fires, as well as for sanitation. A potable water system is needed for proper large scale landfill operations.

Water under pressure is available at 21% of the sites serving less than 10,000 people; whereas, at the larger installations operated by agencies with over 100,000 population, some 52% have water under pressure as noted in Table 5(E).

Telephone communications are common at the larger landfills (52%); whereas, only 13% of the less than 10,000 population landfill operations have available telephones (Table 5(F)).

Methods of Financing.—In contrast to most public utilities, landfills normally need minor capital expenditures for equipment or structures and, hence,

TABLE 6.—LANDFILL FINANCING

No. (1)	Method of Financing Landfills (2)	% of Landfills Using (3)
1.	General tax fund only	57
2.	Collection fees only	11
3.	Collection and dumping fees	11
4.	General tax and dumping fees	10
5.	General tax and collection fees	5
6.	General tax, collection fees and dumping fees	4
7.	Dumping fees only	2

TABLE 7.—LANDFILL DISTANCE TO NEAREST DWELLING

No. (1)	Distance from Landfill to Nearest Dwelling, in feet (2)	% of Landfills (3)
1.	0-250	12
2.	250-500	20
3.	500-1000	26
4.	1000-2640	24
5.	Over 2640	18

relatively few fills appear to have been financed by means of bond issues. In order to obtain sufficient disposal sites to serve future needs it may be necessary to purchase in advance suitable land areas financed by means of bonded indebtedness.

Over 57% of the reporting landfills are supported by general tax funds only. This is a far more popular method than other methods of financing, shown in Table 6, such as: collection fees only; collections and dumping fees; general tax and dumping fees; general tax and collection fees; general tax, collection fees, and dumping fees; and dumping fees only.

Location.—It is generally difficult to locate a new landfill site because adjacent property owners normally are opposed to the establishment of a new site.

The old-fashioned "burning dump" has created an extremely poor public impression that has, unfortunately, been transferred to the sanitary fill. In spite of these factors, there are many landfills established in residential areas with close proximity to dwellings.

The distance of the reporting landfills from the nearest dwelling is enumerated in Table 7. Most of the landfills (88%) are over 250 ft away from the nearest dwelling, although some 12% are operated within less than 250 ft from dwellings.

The majority (55%) of the disposal sites are in rural or suburban areas with the remainder in industrial, residential, and commercial zones numerically described in Table 8.

TABLE 8.—LANDFILL LOCATION ZONE

No. (1)	Fill Location (2)	% of Landfills Using (3)
1.	Residential	19
2.	Industrial	21
3.	Commercial	5
4.	Rural or Suburban	55

TABLE 9.—ONE-WAY HAUL DISTANCE

No. (1)	Length of Haul, in miles (2)	% of Landfills (3)
1.	0-1	10
2.	1-2	32
3.	2-3	17
4.	3-4	14
5.	4-5	10
6.	5-6	7
7.	6-7	4
8.	7-8	1
9.	8-9	1
10.	9-10	1
11.	10-15	2
12.	Over 15	1

Average Distance of Haul (One-Way) to Landfill.—The greatest number (32%) of landfills reported that the length of haul averaged some 1 to 2 miles. Relatively few landfills received average length of hauls greater than 7 miles. Some 1% of the landfills have lengths of hauls greater than 15 miles. Detailed information is shown in Table 9.

The major costs of refuse disposal are incurred in collection and hauling; it is therefore desirable to minimize the length of haul distance. A good economic evaluation for a disposal system will review: (a) the feasibility of increased hauling distance, (b) the use of refuse transfer stations, and (c) the additional costs for alternative, more costly refuse disposal methods.

Public Reaction.—The survey has demonstrated that there are many operating problems at sanitary landfills. Nevertheless, the sanitary landfill procedures are basically so sound and simple that public reaction, in most all cases, has been favorable. Improved planning, engineering, and supervision of operations would eliminate troubles and promote good public relations. For example, special access roadways connected via major thoroughfares are needed to handle large scale, slow-moving refuse collection vehicle traffic entering a landfill; residential streets should not be used for access roads.

As shown in Table 10, sanitary landfills have achieved general acceptance by the public; whereas before starting operation, some 24% reported opposition; after operation was initiated, only 2% have still continued to receive public opposition.

TABLE 10.—PUBLIC ACCEPTANCE BEFORE AND AFTER LANDFILL OPERATIONS

No. (1)	Public Reaction Before Operation Began (2)	% of Landfills (3)
1.	In favor	36
2.	Indifferent	40
3.	Opposed	24
	Public Reaction After Operation	
1.	In favor	74
2.	Indifferent	24
3.	Opposed	2

TABLE 11.—ORIGINAL LAND USE

No. (1)	Original Land Use (2)	% of Landfills (3)
1.	Agriculture	32
2.	Vacant	20
3.	Open dump	11
4.	Excavation	11
5.	Swamp	9
6.	Wasteland	9
7.	Riverbank	4
8.	Industrial	2
9.	Lake	2

Original Land Use.—A common belief is that landfills are largely located in old existing quarries, swamps, or other wasteland sites; however, the desirability of locating a landfill within economic refuse haul distance has resulted in most fills being constructed close-in, within vacant areas or adjoining agricultural land. The transportation savings that accrue may enable a community to pay what might normally be considered an excessive price for a central land site.

The original land use for most landfill sites was agriculture (32%), vacant land (20%), existing dumps and excavations. Common land use conditions prior to landfill operation are shown in Table 11.

Filled Site Use.—Completed landfills may be an extended problem because gases can be generated for long periods of time, continuous settlement occurs, and relatively little is known about the future characteristics of reclaimed land. Experience indicates that a completed landfill can be safely used for parks, golf courses, parking lots, agriculture, airports, as well as certain other restricted purposes. Further technical investigations are needed to ascertain the limiting conditions for both decomposing fill refuse and adapting fill sites for conventional improvements.

The proposed land use for the completed landfills are reported in Table 12. The most popular use for landfills appears to be for recreational purposes including golf courses and parks. The second most popular application is for

TABLE 12.—COMPLETED LANDFILL USE

No. (1)	Land Use (2)	% of Landfills (3)
1.	Undetermined	39
2.	Recreation	26
3.	Industrial	18
4.	Agriculture	9
5.	Homesites	2
6.	Airports	2
7.	Incinerators	1
8.	Schools	1
9.	Refill	1
10.	Levees	.5
11.	Highways	.5

TABLE 13.—SITE ACQUISITION METHODS

No. (1)	Method of Acquisition (2)	% of Landfills (3)
1.	Purchase	36
2.	City Owned	32
3.	Private-Loaned (still privately owned)	18
4.	Leased	13
5.	Condemned	1

industry, particularly as open vehicle parking and storage yard sites (18%). About 39% of the landfills have no planned use, although agricultural, homesites, airports, incinerators, school, refills, levees, and highway construction are also reported to be planned.

Acquisition of Sites.—The acquisition of sufficient disposal sites is the major difficulty faced by most large cities. Metropolitan areas normally do not have available sufficient land for long period refuse disposal. Many communities prefer to go to alternative disposal systems such as incineration rather than to condemn land.

As shown in Table 13, most landfills were acquired by purchase (36%). City owned sites are the second most popular location; whereas, privately

loaned and owned sites are used in some (18%) of the landfills. The leasing of private sites is done at 13% of the locations. Condemnation is seldom practiced and was resorted to at less than 1% of the reported fills.

Life Span.—The average and median estimated months of landfill life remaining is shown in Table 14 for the various population groups served.

It is apparent that there was less than 8 yr remaining capacity (84 months) in the average operating landfill. The average median disposal life for all of the reporting fills is about 5 yr. The limited landfill capacity available at these sites is indicative of minimum planning for future needs. Certainly most utilities would be expected to have greater than 5 yr life expectancy.

TABLE 14.—REMAINING FILL LIFE

No. (1)	Population Group Served (2)	Average Estimated Months of Fill Life Remaining	
		Mean (3)	Median (4)
1.	0-10,000	87	60
2.	10-25,000	73	52
3.	25-50,000	70	36
4.	50-100,000	87	60
5.	Over 100,000	77	48
6.	All Fills	84	60

TABLE 15.—PREDOMINANT NATIVE SOIL TYPES

No. (1)	Type of Soil (2)	% of Landfills (3)
1.	Clay	35
2.	Sand	34
3.	Sand and Clay	18
4.	Clay and Rock	6
5.	Rocky	4
6.	Sand and Rock	2
7.	Sand, Clay and Rocky	1

Types of Soil.—Sanitary landfills are generally located in mixtures of sand and clay type soils because these materials are common and enable satisfactory landfill operations. The various soil types encountered are described in Table 15. Rocky soils are obviously less adaptable for landfill use. Heavy clay type soils may shrink and expand excessively with moisture changes. Cracks that occur in an adobe or clay soil may open fissures that permit rodents and vermin to have access to the buried refuse. Furthermore, these fissures enable combustible and partially stabilized gases to readily escape so that fire hazards within an existing fill can occur.

Depth to Ground Water.—Agencies concerned with ground water pollution control often do not permit construction of landfills directly within an existing ground water aquifer. There is evidence that landfills may contribute some mineralization to the ground water and that industrial wastes may thus pollute

the aquifer. In some areas it is also forbidden to dispose of industrial waste into landfills constructed above an existing ground water table. These landfills may not be in contact with the ground water, but there has been concern as to possible leaching action and subsequent pollution of the water table.

Over 79% of the landfills are reported in Table 16 as located within 20 ft of ground water. Some 27% of the landfills must be operated at or near ground water as shown in the following Table. Ground water pollution by landfills has often been considered a critical item; whereas, the survey indicates that in general site planning there has been minimum concern with ground water pollution. However, as described in another section 6% of the sites also report ground water pollution problems.

TABLE 16.—DEPTH TO GROUND WATER

No. (1)	Depth to Ground Water, in feet (2)	% of Landfills (3)
1.	0-5	27
2.	5-10	20
3.	10-15	15
4.	15-20	10
5.	20-25	5
6.	25-50	13
7.	Over 50	10

TABLE 17.—TYPES OF LANDFILL OPERATIONS

No. (1)	Type of Operation Used (2)	% of Landfills (3)
1.	Trench	38
2.	Ramp	26
3.	Area	22
4.	Trench and Ramp	6
5.	Trench and Area	5
6.	Ramp and Area	2
7.	Trench, Ramp and Area	1

Engineering Planning.—The general inadequacy of engineering planning for most landfills is also demonstrated by survey data describing the: (a) need for costly importation of earth cover, (b) inefficient use of existing landfill disposal capacity, and (c) cost in regrading landfill sites to provide for their ultimate use as golf course or building sites. It is commonplace to physically remove certain existing landfills for new highways or industrial tracts when advanced engineering planning could have reduced these expenses.

Only 9.3% of the landfill sites reported that test ground borings were made prior to fill operations, 14% of the reporting fills have specially engineered drainage devices. The use of hilly and mountainous canyons for landfills particularly necessitates the use of engineered drainage facilities. In contrast, some 92% of the landfills indicate that they have all-weather access roads to the disposal site.

Landfill Disposal Methods.—The application of a particular landfill system is dependent on existing site topography as well as available soil and local experience. In general, the type of disposal methods most suitable should be determined by means of engineering tests and know-how in order to enable optimum disposal practices.

Trench, ramp, and area methods in the sequence described are the most popular for landfill disposal as indicated in Table 17.

Types of Refuse Collection Vehicles.—Most of the landfills reported that compactor type trucks are commonly used to deliver refuse, and only 12% of the landfills stated that non-compacting type vehicles provided their major collection equipment.

Refuse Quantity Measurement.—Accurate determination of the quantity of refuse handled is necessary for dependable accounting records and cost determinations. Landfills that directly charge users for disposal privileges particularly need weighing scales in order to accurately and fairly determine payments.

TABLE 18.—REFUSE QUANTITY MEASUREMENTS

No.	Population Group Served	% of Landfills					
		By weight	By estimated weight	By estimated volume	By weight and estimated weight	By estimated weight and volume	Not determined
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1.	0-10,000	6	11	52	2	2	27
2.	10-25,000	3	10	49	.5	5	33
3.	25-50,000	13	13	52	.5	4	18
4.	50-100,000	17	17	57	.5	.5	9
5.	Over 100,000	24	16	48	4	4	4

About one half of the reporting landfills including even the larger sites reported refuse quantity measurements were made by estimating volume. Accurate weight measurements were made at less than 25% of the landfills as shown in Table 18. Relatively few (4% to 33%) of the sites in the different population classifications did not attempt to determine the weight or volume of the disposed refuse. However, most all of the landfill operations, particularly the larger sites, have attempted some type of refuse quantity measurements.

Compaction Ratio.—Efficient landfill operation implies both the obtaining of optimum compaction of refuse within the landfill and rapid refuse stabilization. There is need for research to determine the parameters that control optimum landfill disposal.

The compaction ratio describes the original volume reduction achieved in the landfill in contrast to the trucked refuse volume. This compaction ratio is reported in Table 19 as being 10:1 at 2 locations, whereas, most sites report landfill volume reduction ratios of between 2:1 to 3:1 as shown. The quantity of the compaction ratio is dependent, of course, on the density of the trucked

refuse and the type of compaction achieved within the refuse collection vehicles. Further indications are that the extent of initial compaction achieved is related to the compaction procedures including addition of moisture and soil to the landfill.

Landfill Density.—The landfill in-place refuse density was reported in pounds per cubic yard. The extremely wide range described in Table 20 indicates

TABLE 19.—COMPACTION RATIO

No. (1)	Compaction Ratio = Trucked volume / In Place Volume (2)	No. of Landfills Reporting (3)
1.	1:1	1
2.	2:1	22
3.	3:1	25
4.	4:1	8
5.	5:1	4
6.	6:1	3
7.	7:1	1
8.	8:1	1
9.	9:1	1
10.	10:1	2

TABLE 20.—INITIAL FILL DENSITY AFTER COMPACTION

No. (1)	Initial Density, in lb per cu yd (2)	No. of Landfills (3)
1.	250	1
2.	300	1
3.	325	1
4.	400	2
5.	450	2
6.	475	1
7.	500	1
8.	600	1
9.	700	2
10.	750	1
11.	800	2
12.	1000	4
13.	1200	1
14.	1250	1
15.	1500	4
16.	1800	3
17.	1900	1
18.	2000	4

operating controls. The reported densities are from 250 lb per cu yd to 2000 lb per cu yd in contrast to normal soil densities that are about 2700 lb per cu yd. The wide variation in filled compaction achieved clearly indicates that improved operating procedures at many sites would provide for a greater fill life.

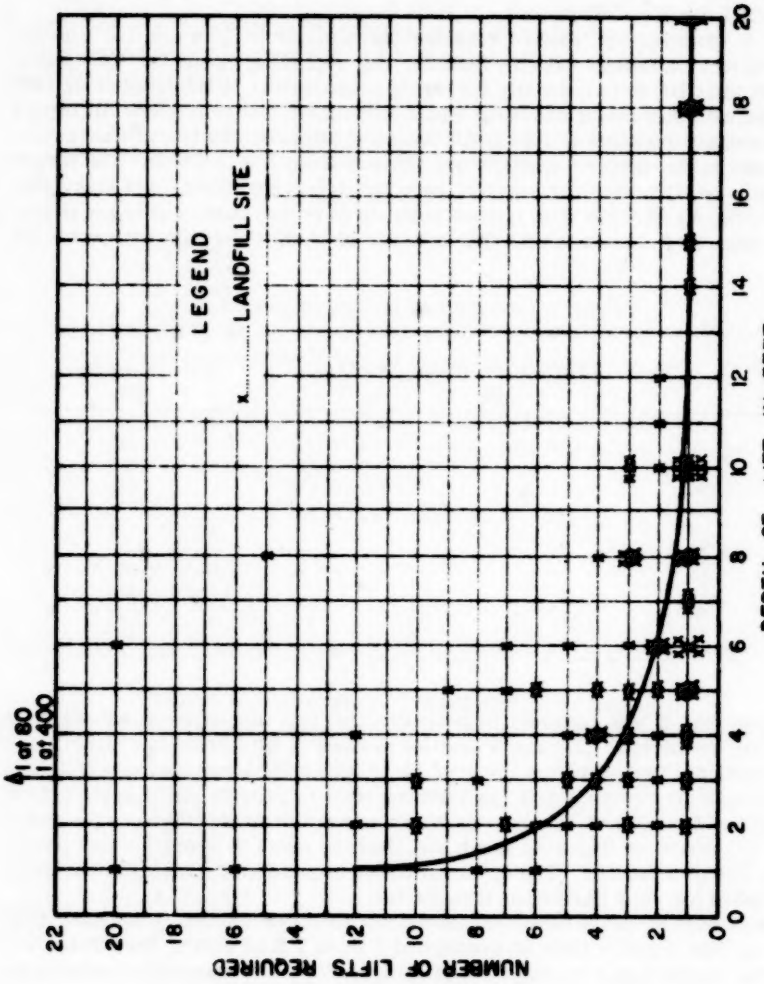

FIG. 1.—NUMBER OF LIFTS AND DEPTH OF EACH LIFT

Landfill Lift Depth.—A feasible method for increasing landfill capacity is to increase its depth. In many areas there is prejudice against forming "an Indian mound;" however, particularly in flat lands, the construction of an elevated park site can be decorative. Hilly golf courses and parks have character. Good public relation practice should provide for construction of earthen dykes, tree plantings, or other types of screens to prevent public observation of routine landfill disposal operations.

Fig. 1 presents the plotted data for the number of lifts and the average depths of the individual lifts in feet, for the reporting landfills. The initial depth of the fills were normally inversely proportional to the number of lifts required, all other conditions being equal. Obviously, where shallow lift depths were used, many layers or lifts of fill were used to bring the site to final grade. The range in the depth of landfills per lift was from 1 ft to 400 ft. The larger fill depths per lift were the result of locations involving hillside and steep embankments. As shown in Fig. 1, most landfills have less than 10 lifts and depths of less than 20 ft for each lift. The success of the deeper fills with each lift

TABLE 21.—DAILY EARTH COVER DEPTH

No. (1)	Depth of Daily Earth Cover, in inches (2)	No. of Landfills (3)
1.	2	8
2.	3	8
3.	4	18
4.	6	61
5.	8	7
6.	9	1
7.	10	4
8.	12	29
9.	18	18
10.	20	1
11.	24	13

indicates that it was possible to conserve the soil and achieve satisfactory operation with deeper fills and a smaller number of lifts prior to reaching finish elevation. The problems involved in constructing large uncompartmented fills include fire control and the sanitary need to provide daily earth cover.

Depth of Daily Earth Cover.—Earth cover requirements influence the quantity of excavated or imported earth and, thus the costs of operation and possibly the life of a landfill. The optimum cover requirement should be carefully established for each particular installation.

The amount of earth cover utilized daily to cover operating sanitary fills was reported to range from an average of 2 in. to 2 ft of earth. (See Table 21) Some 61 landfills report that 6 in. of daily earth cover is used at their sites. Other popular earth cover depths are 4 in., 12 in., 18 in., and 24 in.

Landfills Requiring Hauled-In Earth Cover.—As has been pointed out previously, it is generally costly and wasteful to haul in earth cover.

Most operating landfills reported that they obtained necessary earth cover material from the site; the percentages of landfills requiring imported cover, as shown in Table 22, ranged from 15% to 32%. The larger landfills serving

populations of over 100,000 had greater need for imported cover (32%) in contrast to the 0-10,000 population sites at which only 15% used hauled-in cover material.

Landfills Permitting Burning.—Burning at landfills was not permitted in most major sites; whereas, it was common (52%) at smaller landfills serving populations less than 10,000. A detailed summary of the landfills permitting burning in relation to the population served is shown in Table 23. The general trend definitely appears to be toward abolition of odoriferous burning operations at landfills.

TABLE 22.—LANDFILLS REQUIRING HAULED-IN EARTH COVER

No. (1)	Population Served (2)	% of Landfills (3)
1.	0-10,000	15
2.	10-25,000	22
3.	25-50,000	18
4.	50-100,000	26
5.	Over 100,000	32

TABLE 23.—LANDFILLS PERMITTING BURNING

No. (1)	Population Served (2)	% of Landfills (3)
1.	0-10,000	52
2.	10-25,000	43
3.	25-50,000	36
4.	50-100,000	37
5.	Over 100,000	16

TABLE 24.—LANDFILLS PERMITTING SCAVENGING

No. (1)	Population Served (2)	% of Landfills (3)
1.	0-10,000	15
2.	10-25,000	17
3.	25-50,000	29
4.	50-100,000	29
5.	Over 100,000	20

Landfills Permitting Scavenging.—Scavenging is an unpopular operation and was not permitted at most landfill sites. As shown in Table 24, less than 30% of all the reporting landfills in various categories still permit scavenging operations. In contrast, Table 25 indicates that salvage operations are only slightly more popular. The larger governmental units more often (48%) allowed salvage than did the smaller landfills (36%). Similarly, in the larger population centers, private salvage by contract or otherwise is more popular (49%);

whereas, in smaller landfill sites, municipal salvage operations are engaged in about as often as private salvage operations (19% and 17%, respectively).

Landfill Operating Problems.—Blowing papers can best be controlled by means of both permanent and temporary fencing, wetting with water to increase the weight of the paper and, obviously, by good housekeeping practices such

TABLE 25.—SALVAGE OPERATIONS

No.	Population Served	% of Landfills		
		Salvage Not Allowed	Salvage by Municipality	Private Salvage by Contract or Otherwise
(1)	(2)	(3)	(4)	(5)
1.	0-10,000	64	19	17
2.	10-25,000	65	11	24
3.	25-50,000	66	9	25
4.	50-100,000	63	3	34
5.	Over 100,000	49	2	49

TABLE 26.—LANDFILL OPERATING PROBLEMS

No. (1)	Problem (2)	% of Landfills (3)
1.	Blowing Paper	46
2.	Flies	34
3.	Fires	27
4.	Rodents	23
5.	Odors	19
6.	Dust	14
7.	Water Pollution	6
8.	Traffic	6

TABLE 27.—WORKING HOURS PER WEEK

No.	Population Served	Hours Per Week			
		Minimum	Mean	Median	Maximum
(1)	(2)	(3)	(4)	(5)	(6)
1.	0-10,000	3	41	46	60
2.	10-25,000	8	48	50	112
3.	25-50,000	36	50	54	144
4.	50-100,000	36	54	63	168
5.	Over 100,000	40	58	70	168

as daily clean-up. Flies and rodents are a nuisance that should not exist at a properly operated sanitary landfill. Numerous investigations have demonstrated that with suitable earth cover of refuse, flies, rodents, and odors will not occur. Water pollution hazards and traffic problems can be controlled by good engineering.

Still, the most often (46%) reported landfill operating problem is that of blowing paper. Surprisingly enough, other common nuisances include flies, fires, rodents, odors and dust. Water pollution was described as an operating difficulty at some six locations. A detailed summary of these operating problems is shown in Table 26.

Working Hours.—The larger sites operated a greater number of working hours than the smaller fills, as shown in Table 27. The reporting installations were open from a minimum of 3 hrs per week to a maximum of 168 hrs per week.

Special winter landfill practices are required at 33% of the reporting sites as shown in Table 28. Other landfills do not require any special winter practices such as earth stock-piling or additional special disposal operations.

TABLE 28.—SPECIAL WINTER OPERATIONS PRACTICED

No. (1)	Population Served (2)	No. of Landfills (3)	% of Landfills (4)
1.	0-10,000	13	27
2.	10-25,000	17	27
3.	25-50,000	21	38
4.	50-100,000	11	31
5.	Over 100,000	6	24
6.	All Fills		33

TABLE 29.—LANDFILLS OPEN FOR UNLOADING AFTER DISPOSAL EQUIPMENT WORKING HOURS

No. (1)	Population Served (2)	No. of Landfills (3)	% of Landfills (4)
1.	0-10,000	11	23
2.	10-25,000	15	24
3.	25-50,000	17	31
4.	50-100,000	12	34
5.	Over 100,000	6	24
6.	All Fills		26

TABLE 30.—SITE FENCING

No. (1)	Population Served (2)	No. of Landfills (3)	% of Landfills (4)
1.	0-10,000	24	50
2.	10-25,000	35	56
3.	25-50,000	23	42
4.	50-100,000	19	54
5.	Over 100,000	9	36
6.	All Fills		49

Most landfills are not open after working hours. As shown in Table 29, only 26% of the total landfills are open for refuse disposal after landfill equipment working hours.

Fencing.—Fencing is not only a useful device to control wind blown papers but is also needed to prevent intrusion of outsiders. Nevertheless, as described in Table 30, only 49% of the reporting landfills were fenced and surprisingly

TABLE 31.—ANNUAL PER CAPITA OPERATING COSTS

No. (1)	Population Served (2)	Dollars Per Capita			
		Minimum (3)	Mean (4)	Median (5)	Maximum (6)
1.	0-10,000	.33	.68	.69	1.20
2.	10-25,000	.25	.50	.50	1.00
3.	25-50,000	.13	.36	.41	1.16
4.	50-100,000	.11	.34	.41	.96
5.	Over 100,000	.07	.20	.21	.50

TABLE 32.—ANNUAL AREA CONSUMED

No. (1)	Population Served (2)	Acres per 10,000 Population	
		Mean (3)	Median (4)
1.	0-10,000	1.5	2.0
2.	10-25,000	0.87	1.0
3.	25-50,000	0.6	0.77
4.	50-100,000	0.45	0.55
5.	Over 100,000	0.30	0.40

TABLE 33.—GOVERNMENTAL REGULATIONS OF LANDFILLS

No. (1)	Population Served (2)	City		County	
		No. of Landfills (3)	% of Landfills (4)	No. of Landfills (5)	% of Landfills (6)
1.	0-10,000	29	60	5	10
2.	10-25,000	38	60	10	16
3.	25-50,000	33	60	13	24
4.	50-100,000	17	49	7	20
5.	Over 100,000	16	64	4	16

enough, only 36% of the larger landfills serving over 100,000 population were properly fenced.

Annual Operating Costs.—The low cost of sanitary landfill operations is confirmed by the experience at many existing fills. Because of this cost advantage, it may be justifiable to budget additional funds for improving operations in order to increase the life expectancy of the fill and the reclamation of the completed landfill for other purposes.

The per capita annual operating costs are reviewed in Table 31. The larger landfills have definitely lower operating costs than do the smaller sites. The less than 10,000 population group fills reported a minimum-maximum range of $0.33 to $1.20; whereas, the over 100,000 population group had a minimum-maximum annual operating cost per capita of $0.07 to $0.50, respectively.

Landfill Acreage Consumption.—The larger landfills reported that they required from 0.3 to 0.4 acres per 10,000 population per yr, as shown in Table 32. In contrast, the less than 10,000 population fills required 5 times as much area. It is believed that this is due to the larger fills being operated both more

TABLE 34.—LANDFILL TECHNICAL PLANNING ADVICE RECEIVED FROM

No.	Population Served	Local City Health Dept.		County Health Dept.		State Health Dept.		Private Consultants	
		No.	%	No.	%	No.	%	No.	%
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1.	0-10,000	8	17	10	21	30	62	7	15
2.	10-25,000	21	33	12	19	39	62	6	10
3.	25-50,000	16	29	14	25	29	53	1	2
4.	50-100,000	11	31	7	20	13	37	2	6
5.	Over 100,000	11	44	3	12	7	28	2	8

TABLE 35.—LANDFILLS HAVING MASTER PLANS

	Population Served	No. of Landfills	% of Landfills
1.	0-10,000	14	29
2.	10-25,000	13	21
3.	25-50,000	19	35
4.	50-100,000	11	31
5.	Over 100,000	5	20

TABLE 36.—FUTURE ESTIMATED SUITABLE LANDFILL SITES DISPOSAL CAPACITY AVAILABLE (IN YEARS)

	Population Served	Median	Mean
1.	0-10,000	10	16
2.	10-25,000	10	17
3.	25-50,000	10	17
4.	50-100,000	11	19
5.	Over 100,000	10	19

efficiently and with greater depths in order to conserve the limited available land.

Landfill Governmental Regulations.—Most landfills (over 70%) as shown in Table 33, operated under some type of municipal or county regulation. Local municipal regulations were some 2-1/2 to 6 times as common as County control.

Landfill Technical Planning Aid.—Landfill technical planning advice is most commonly provided by the State Health Department for the smaller sites and the local City Health Department for the larger fills. Private consultants were used at 2% to 15% of the installations serving various populations. Table 34 contains detailed data concerning technical planning.

Less than 35% of the landfills had an initial master plan. Surprisingly, the larger population centers of over 100,000 persons are most deficient with only 20% requiring a master plan for their operation. The need for improved technical planning is evident.

Future Landfill Site Disposal Capacity.—The estimated life of lands available for sanitary fill operations is shown in Table 36. The median landfill availability for all the five categories of population served was 10 yr to 11 yr. The statistical information is surprisingly uniform in light of the fact that large population centers have commonly reported a lack of sufficient available landfill sties. Planning experience indicates that by judicious use of transfer stations and carefully developed land reclamation projects, landfill sites can be and are being found to serve at least a portion of most metropolitan refuse disposal requirements.

CONCLUSIONS

Some 251 landfill sites have reported on their technical operations. The wide variation in their operating practices has indicated that additional engineering and planning would conserve funds, available lands and reduce present and future operating problems. The technical report has been presented in outline form and should be referred to as a detailed summary containing both direct and implied conclusions.

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DISCUSSION

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STORM WATER AND COMBINED SEWAGE OVERFLOWS[a]

Discussion by H. G. Aronson

H. G. ARONSON,[5] A.M. ASCE.—New York City's water enthusiasts could look forward to a future with fewer beach pollution hazards, if the authors' proposals were adopted, such as, "the use of retention basins for the chlorination and the removal of floating solids from the intermittent discharges of combined sewage overflows." But several questions require consideration before a final decision is reached.

Investigation of Diversion Chamber with Non-moving Parts.—New York City's combined sewer system intercepts the dry-weather sanitary flow by means of chambers with moving parts. Because of the moving parts, the chambers are subject to heavy maintenance and stoppage from solids carried by raw sewage.

If the stoppage occurred with the chamber gates in an open position, then the retention basin would receive only a portion of the storm flow, with the remainder intercepted, carried to, and possibly overloading the treatment plant, or bypassing the plant without any treatment. Thus, in effect, raw sewage discharge would be eliminated at the combined sewer outfall, but it would continue at the treatment plant outfall.

If the diversion chamber stoppage occurred with the gate in a closed position, then the retention basin pump would require continuous operation during dry weather flow, and there would be an undesirably high degree of raw sewage settling in the retention basin.

To date (1961), limited information of design value is available on control-devices without moving parts. In view of the large expenditure necessary for the construction of retention basins, this might be an appropriate time for New York City to enlarge the studies on control devices without moving parts.

Location of the Retention Basins.—From an esthetic point of view, it may be necessary to locate the basins undesirably close to residential and recreation areas. This proximity could be one of the reasons why the sites may have originally been rejected as locations of the treatment plants.

The authors suggest a chlorine contact period of 15 min. A reduction to 10 min, in conjunction with a higher chlorine dosage would provide adequate chlorination,[6] and would permit the reduction of basin sizes to more adaptable dimensions.

High Tides.—The affect on the retention basin operation of extreme high tides in combination with maximum storm runoff should be investigated.

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[a] January 1961, by Samuel A. Greeley and Paul E. Langdon (Proc. Paper 2718).

[5] Hydr. Engr., E. Lionel Pavlo, New York, N. Y.

[6] "Chlorination Of Mixed Sewage And Storm Water," by T. R. Camp, Proceedings, ASCE, Vol. 87, No. SA 1, January, 1961.

Would the additional cost of basins capable of accommodating these rare occurrences be warranted?

Alternate Solutions.—Other proposals could be:

a. Extending the combined sewer outfall so as to prevent sewage effluent from reaching bathing areas in dangerous concentrations.

b. Huge underwater septic tanks—real estate problems would be reduced, construction costs would be higher, and maintenance costs would be lower. Surface scum would be removed at regular intervals. Chlorination should be given at the septic tank effluent at the beginning of an outfall of a length sufficient to provide contact, or, in the second of two compartments.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

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c. Discussion of several papers, grouped by divisions.

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